McMahon & Mann
Consulting Engineers, P.C.

GEOTECHNICAL ENGINEERING REPORT
TONAWANDA CREEK ROAD
SLOPE STABILIZATION PROJECT

TOWN OF CLARENCE,
ERIE COUNTY, NEW YORK

Prepared for:

Erie County Department of Public Works
45 Oak Street
Buffalo, New York 14203

By:

McMahon & Mann Consulting Engineers, P.C.
2495 Main Street
Buffalo, New York 14214

November 2004
File: 04-013

McMahon & Mann 2495 Main Street, Suite 432 • Buffalo, New York 14214
Consulting Engineers, P.C.

Donald R. McMahon, P.E.
Michael J. Mann, P.E.
Thomas R. Heins, P.E.
Kenneth L. Fishman, Ph.D., P.E.
James Bojarski, P.E.
Shawn W. Logan, P.E.
Andrew J. Nichols, P.E.
Todd W. Swackhamer, P.E.

November 30, 2004
File: 04-013

Mr. Brian Kirby, P.E.
Erie County Dept. of Public Works
45 Oak Street
Buffalo, New York 14203

RE: Geotechnical Engineering Report,
Tonawanda Creek Road Remediation,
Town of Clarence
Erie County, New York

Dear Mr. Kirby;

Enclosed are five copies of our report for the Tonawanda Creek Road remediation project. The report presents the results of our subsurface exploration and testing program, our recommendations for remedial design and estimated costs for the remediation.

We look forward to continuing to work with the Erie County Department of Public Works on this interesting project.

Sincerely yours,

McMAHON & MANN CONSULTING ENGINEERS, P.C.

Michael J. Mann, P.E.

cc: Mr. David F. Pratt, P.E. Greenman-Pedersen, Inc.

Telephone: 716-834-8932 • Fax: 716-834-8934

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GEOTECHNICAL ENGINEERING REPORT
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

I. INTRODUCTION

A slope failure along the south bank of Tonawanda Creek has damaged a portion of Tonawanda Creek Road between Transit Road and Westphalinger Road in the Town of Clarence (see Figure 1 for location plan). A section of Tonawanda Creek Road, approximately 250 feet long has dropped about 10 feet and pushed soil into Tonawanda Creek. The failure area has continued to expand laterally, encompassing more of the road since it was discovered on June 25, 2004.

The Erie County Department of Public Works (ECDPW) required subsurface information in the vicinity of the failure area, conceptual remedial design recommendations and estimated costs for the remediation. To accomplish this, MMCE:

- Engaged Earth Dimensions, Inc. (EDI) to make four test borings,
- Monitored the test borings, reviewed the test boring logs prepared by EDI and measured the strength of the soil in the boreholes,
- Monitored installation of a standpipe piezometer to measure the groundwater level and an inclinometer to measure the slope movement,
- Reviewed the soil samples collected from the test borings and laboratory tested selected soil samples, and
- Prepared this report that summarizes the subsurface conditions and presents our recommendations for designing the remediation of the road.

Section II describes data collected for this project and Section III describes the surficial and subsurface conditions. Our opinion regarding the cause of the failure is presented in Section IV and Section V presents our recommendations for remediation of the road and comparative cost estimates.

II. DATA COLLECTION

A. Subsurface Explorations

EDI made four test borings, designated as Bore 1-04 through Bore 4-04. EDI made Bores 1-04 and 2-04 on the east side of the failure area from the existing road grade and Bores 3-04 and 4-04 on the west side of the failure also from the existing road grade. On each side of the failure area, EDI extended one boring to bedrock (Bore 1-04 and Bore 3-04) and sampled it continuously including completing Standard Penetration Tests (SPT's) and coring rock. Figure 2 is a plan showing the existing site conditions and the location of the subsurface explorations.

EDI collected split spoon soil samples and thin walled tube samples of the overburden soils and core samples of the rock. Appendix A includes a description of the test boring methods and copies of the test boring logs.

B. Vane Shear Tests

MMCE reviewed the results of each boring with EDI and selected depths for insitu vane shear tests (VST's). EDI advanced a second boring at each location (Bores 2-04 and 4-04) for vane shear testing and to collect thin walled Shelby tube samples. MMCE measured the shear strength of the soil within the boreholes using the VST. Appendix B presents a description of the methods used to make these measurements and the data collected.

C. Inclinometer and Piezometer Installation and Monitoring

EDI installed an inclinometer in Bore 1-04 on the east side of the failure area. The inclinometer consists of a plastic casing that is grouted into rock. This instrument provides information about the depth of the failure zone and whether or not movement is continuing. Details of the inclinometer installation are presented on the boring logs in Appendix A.

EDI installed a standpipe piezometer adjacent to Bore 3-04 and 4-04 on the west side of the failure area. The piezometer consists of a plastic casing that is slotted allowing measurements of the groundwater level with time. Details of the piezometer installation are presented on the boring logs in Appendix A.

MMCE monitored the inclinometer and piezometer on several dates after they were installed. The monitoring data are presented in Appendix C and indicate no significant lateral movement has occurred since the inclinometer was installed. Groundwater measurements are summarized in Appendix C.

D. Laboratory Testing

MMCE measured the moisture content of soil samples and engaged Geotechnics of Pittsburgh, Pennsylvania to measure the unit weight, shear strength and Atterberg Limits of thin walled samples. Appendix D presents the laboratory test methods and the test results.

MMCE also engaged Geotesting Services, Inc. (Geotesting) of Totowa, New Jersey to test Shelby tube samples of the soft clay. Geotesting measured the natural moisture content, Atterberg limits gradation and strength of the soft clay soils using the laboratory vane test. The test procedures and results are presented in Appendix D.

In addition to testing the soft clay, MMCE requested that Geotesting add dry cement and a combination of dry cement and lime to the soft clay, allow the mixture to cure and then measure the compressive strength of the mixture after 7, 14, 28 and 56 days. We requested this testing to evaluate the potential stabilizing effect of mixing cement or cement and lime with the soft clay to create a stable base for reconstruction of the road embankment. The mixing and test procedures and the test results for the samples that were cured for seven days are included in Appendix D. The dry mix remedial method and the dry mixing test results are discussed in more detail in Section V.

III. SITE CONDITIONS

A. Background Information for Tonawanda Creek

1. General

Tonawanda Creek flows in a generally westerly direction from its headwaters east of Batavia to its discharge into the Niagara River at Tonawanda, New York. Figure 3 is a plan showing a section of the western portion of Tonawanda Creek. The western portion of Tonawanda Creek, from the Niagara River to Pendleton, is part of the present day Erie Barge Canal. At Pendleton the Erie Canal turns north toward Lockport and Tonawanda Creek extends eastward and begins its meandering orientation.

Tonawanda Creek was part of the Erie Canal prior to the Canal's present path from Lake Erie through the Black Rock Canal and along the Niagara River into Tonawanda Creek. Prior to about 1920, the Erie Canal followed a path through the City of Buffalo along the present day alignment of the New York State Thruway, then through Tonawanda and joined Tonawanda Creek near its confluence with Ellicott Creek. A dam, approximately 5 feet high, was constructed across Tonawanda Creek just downstream of its confluence with Ellicott Creek. The dam increased the water level in Tonawanda Creek and the Erie Canal reducing the amount of excavation necessary to create this portion of the Canal.

When the Erie Canal was abandoned through the City of Buffalo in the 1920's and realigned to its present configuration, the dam was removed and Tonawanda Creek was lowered to meet the Niagara River.

2. Creek Slope Failures

The geologic conditions along Tonawanda Creek have led to a history of bank failures along the creek. The western portion of Tonawanda Creek flows through soft clay soils that were deposited during the last glacial recession. As the glacial ice retreated, a glacial lake (Lake Warren) was impounded between the glacial ice and higher ground to the south. Soft clay was deposited on the bottom of the glacial lake and remained after the glacial ice and lake waters receded. Subsequent glacial deposits resulted in a layer of sand and silt above the soft clay along the banks of Tonawanda Creek.

As shown on Figure 3, Tonawanda Creek meanders as it flows from east to west. This flow pattern results in erosion of the stream bank on the outer bend in the creek and deposition on inside bends of the creek. Erosion of the outer creek banks over time results in a loss of support for the banks. This effect, coupled with saturation of the upper silty soils due to rain events and poor drainage, apparently is the cause for many of the slope failures along Tonawanda Creek.

Figure 3 shows the location of several sites along Tonawanda Creek, and along Ransom Creek, where slope failure problems have occurred in the recent past or are presently occurring. These include the Block Church Road Site, where the US Army Corps of Engineers is currently stabilizing several hundred feet of the creek bank using a sheetpile wall and heavy riprap, an approximately 400 foot long failure just east of Rapids near a private residence, a failure near Campbell Boulevard and several locations along Hopkins Road where the banks of Ransom Creek are failing. Discussions with local landowners indicate that numerous other failures have occurred in the past.

Three of the sites shown on Figure 3 have been remediated by ECDPW in the past several years. Site #1 was remediated by excavating about 10 feet into the soft clay and constructing a reinforced earth wall to support the road embankment (see Figure 4 for typical section). Site #2 and the Burdick Road Site were remediated by placing riprap to support the embankment toe and slope as shown on Figures 5 and 6. We visited these three sites and found that Tonawanda Creek Road at Sites #1 and #2 appears to be in good condition. Burdick Road however is showing signs of continued slope movement such as slumping and cracking near the top of the slope.

B. Surficial Site Conditions

At this site, Tonawanda Creek Road is oriented approximately in an east-west direction, (see Figures 2 and 3). The area where the road has collapsed is on an outside bend in the creek similar to other failure locations along the creek. The collapsed area is approximately 250 feet long however the affected area extends a few hundred feet beyond each side of the collapse area. The photos on Figure 9 show the failure area.

The road surface in the portion of the road that has not failed is approximately elevation (El.) 587 feet (178.9 meters). The scarp in the failure area is about 10 feet (3.1 meters) high and the face consists of silty sand to sandy silt. MMCE observed water seeping out of the face of the scarp at several locations (see photos on Figure 10). Ponded water, in the area south of the failure area, could be infiltrating into the ground and seeping out of the face of the scarp.

The area between the road and the creek is about 40 to 60 feet (12.1 to 18.3 meters) wide and ranges in elevation from about El. 575 feet (175.3 meters) to about El. 570 feet (173.7 meters). This area was pushed out into the creek during the slope failure creating the bulge shown on Figure 2. The area is vegetated with brush, grass and trees. Several tree trunks lean back into the slope indicating ground movement at their bases.

On September 23, 2004, the day of the site survey, the creek was at El. 568 feet (173.2 meters). We observed a mound of soil pushed up into the creek as shown on the photos in Figure 11. The ground contours on Figure 2 show that the creek bottom is mounded up in the vicinity of the failure to about El. 565 feet (172.2 meters) and that the creek is constricted in this area. Upstream and downstream of the failure area, the low point in the creek bed is about El. 560 feet (170.7 meters).

C Subsurface Conditions

1. General

Figures 7 and 8 depict the soil types and depths observed in Bores 1-04 and 3-04 including the results of borehole and laboratory testing completed to date. As shown on Figures 7 and 8, test borings Bore 1-04 and Bore 3-04 show a similar soil sequence. In general, a few feet of fill is present at each boring location. Beneath the fill, a silty sand deposit was observed to a depth of about 10 feet (3.04 meters). A soft silty clay deposit was observed from a depth of about 10 feet (3.04 meters) to about 35 feet (10.67 meters) in the borings. The soft clay covers a glacial till deposit (a mixture of gravel, sand, silt and clay). Top of bedrock was observed at a depth of about 68 feet (20.73 meters).

The following paragraphs describe the properties of the soil and rock observed in the borings. Refer to the logs in Appendix A for additional details regarding the overburden and bedrock stratigraphy. Depths described in the following sections are relative to the existing Tonawanda Creek Road pavement.

2. Fill

Bores 1-04 and 3-04 were made through the existing Tonawanda Creek Road pavement and encountered sand fill to a depth of 2 feet (0.61 meters) underlying the asphalt pavement.

3. Sand/Silty Sand

Beneath the sand fill a layer of sand, sandy silt or silt was observed in Bores 1-04 and 3-04. The silt and sand deposit is about 6 feet (1.8 meters) thick and extends to a depth of 8.5 to 9 feet (2.6 to 2.7 meters).

4. Silty Clay

A soft to very soft silty clay deposit was observed in the borings from a depth of about 10 feet (3.1 meters), El. 577 feet (El. 175.9 meters), to about 35 feet (10.7 meters), El. 552 feet (El. 168.3 meters) deep. The characteristics of the soft silty clay deposit are significant relative to the road failure. As shown on Figures 7 and 8, the SPT N-values range from WR (weight of rods) to WH (weight of hammer), signifying that the weight of the drilling rods or the weight of rods and hammer in the borehole was sufficient to advance the split spoon sampler the specified 24-inch distance.

The VST measured a peak shear strength that ranges from 228 to 608 pounds per square foot (psf). The remolded undrained shear strength at the same locations varies from 35 psf to 186 psf. The ratio of the peak to the remolded strength varies from about 2.5 to 10 indicating that the soft silty clay is slightly to moderately sensitive. As summarized in Appendix D, laboratory strength measurements made using a laboratory vane on Shelby tube samples are similar to the measurements made using the field vane.

The fact that the soft clay is sensitive, means that when it is disturbed it has a tendency to lose strength. As discussed in Appendix D, the water content and Atterberg Limit test data also indicate that the clay is sensitive.

5. Glacial Till

A glacial till deposit underlies the soft silty clay from a depth of about 35 feet (10.7 meters) to the top of rock at a depth of about 68 feet (20.7 meters), El. 519 feet (158.2 meters). The soil in this deposit consists of a mixture of gravel, sand, silt and clay of proportions that vary from location to location. The glacial till deposit is soft or loose for about the upper 5 feet (1.5 meters), then becomes dense or hard based on the SPT N-values that generally range from about 40 to more than 100. Boulders and cobbles were encountered in the glacial till, the presence of which could inflate the SPT N-values.

6. Bedrock

Dolomitic shale bedrock of the Camillus Formation underlies the glacial till. The rock core samples are medium hard to hard with gypsum deposits and horizontal and low angle fractures. The rock quality designation of the samples (see Appendix A for definition) varies from 35 percent to 71 percent.

7. Groundwater

The groundwater level in the soft clay soil is expected to coincide approximately with the top of the soft clay deposit at about El. 577 feet (El. 175.9 meters). Perched groundwater is present in the sand and silts above the soft clay as indicated by the water observed seeping out of the face of the failed slope. The perched groundwater level in the sand and silt is from infiltration that varies throughout the year depending on rainfall. Groundwater level measurements in the piezometer in Bore 4-04 are presented in Appendix A and vary from dry to a depth of 7 feet (2.13 meters), El. 580 feet (El. 176.8 meters) or about 3 feet above the top of the soft clay.

D. Tonawanda Creek Hydrology

The United States Geologic Survey (USGS) maintains a gaging station at Rapids, approximately 4 miles upstream of the site. The gaging station provides stream flow and elevation data from 1865 through the present. The following table is a summary of the maximum creek elevation and flow at the Rapids gaging station for the past 150 years.

Largest Event in Previous	Gage Height at Rapids	Creek Elevation at Rapids (ft.)	Creek Flow at Rapids (cfs.)	Estimated Creek Elevation at the Site (ft.)	Creek Cross Sectional Area (sq. ft.)	Estimated Average Flow Velocity (ft./sec)
1 year	13.74	584.93	4990	579.79	1827.3	2.73
5 years	13.74	584.93	4990	579.79	1827.3	2.73
10 years	15.33	586.52	6600	581.38	2151.6	3.07
20 years	16.38	587.57	8500	582.43	2373.7	3.58
50 years	16.96	588.15	10600	583.01	2499.2	4.24
100 years	17.50	588.69	12000	583.55	2655.2	4.52
150 years	18.90	590.09	20000	584.95	3001.5	6.66
Yearly mean	2.53	573.72	410	568.58	224.6	1.83

On September 23, 2004, the date of the Tonawanda Creek site survey, the elevation of Tonawanda Creek at the Rapids gaging station was 5.14 feet (El. 1.6 meters) higher than the elevation at the site. We estimated the creek elevation at the site by subtracting this difference from the gaging station elevation as summarized on the table. Additionally, we estimated the average flow velocity at the site by dividing the flow measured at the gaging station by the cross sectional area of the creek at the site.

As indicated on the summary table, each year, the elevation of Tonawanda Creek gets at least as high as about El. 580 feet (El. 176.8 meters), flooding the ground between the road and the creek to within several feet of the Tonawanda Creek Road surface.

IV. CAUSE OF ROAD FAILURE

Photos taken on June 25, 2004 (Figures 12 and 13) show that the failure started as a relatively small failure involving the northern shoulder of the road. In our opinion, a build up of water pressure in the silty sand deposit overlying the soft clay likely led to the initial failure. Evidence of water seeping out of the silty sand could still be observed days after the failure, as shown on Figure 10.

We completed slope stability analyses to evaluate the initial slope failure condition. The analysis depicted on the "Original Conditions – Shallow Failure," (Appendix E) includes a shear strength for the soft clay of 350 pounds per square foot (17.0 kPA), approximately the average of the peak strength values measured in the VST's and the laboratory vane tests. The analysis also considers the groundwater level in the silty sand deposit to be at the ground surface. The analyses for this condition indicate a factor of safety less than unity. This supports the hypothesis that a build up of water pressure in the silty sand deposit led to the shallow failure initially observed at the site.

As evidenced by the laboratory and field test data, the soft clay is sensitive and loses strength when it is disturbed. In our opinion, the initial failure disturbed the deeper soft clay beneath the failure area. We believe that this disturbance caused a loss of strength leading to the larger failure that encompassed the entire road.

We completed slope stability analyses to evaluate the deeper failure. The analysis depicted on "Original Conditions – Deep Failure," includes a shear strength for the soft clay of 92 pounds per square foot (4.4 kPA), approximately the average of the remolded strength values measured in the VST's. The analysis for this condition indicates a factor of safety less than unity, demonstrating that after the soft clay soils were disturbed, they did not have sufficient strength to support the road embankment.

The failure limits shown on the analysis coincide with the location of the scarp and the toe of the failure observed in the field. As indicated on the analysis section in Appendix E, the analysis indicates that the failure extends to the bottom of the soft clay layer.

V. DESIGN RECOMMENDATIONS AND COMPARATIVE COST ESTIMATES

A. General

MMCE reviewed the various methods available to stabilize moving earthen slopes and concluded that the three methods applicable to this site include constructing a soil retaining wall, strengthening the soil and flattening the creek banks and relocating the road.

The remediation options must address two objectives: providing a stable base upon which to reconstruct the road embankment and protecting the bank from future erosion. The remediation options presented below meet both of these objectives.

We considered placing riprap or a reinforced earth wall to support the toe and slope of the road embankment as done at Sites #1 and 2 and at the Burdick Road site. The analysis depicted on "Rip-Rap Remediation Analysis" in Appendix E indicates a factor of safety less than unity for failure surfaces extending beneath the riprap or reinforced earth zone. Considering the extent of the soft disturbed clay soils at the site, in our opinion this remediation approach will not provide sufficient support for a new road embankment.

Three remediation approaches are considered feasible for this site. These include: containing the soft clay with an anchored sheet pile wall, reinforcing the soft clay soil beneath the road embankment with cement or a combination of cement and lime using the dry soil mixing technique and flattening the creek banks and relocating the road to the south. Restoration and protection of the creek bank is necessary for each option. A discussion of each alternative follows:

1. Anchored Sheet Pile Wall

An anchored sheetpile wall such as that shown on Figures 14 and 15 could be installed to retain the soft silty clay soil and allow reconstruction of the road. The wall must extend through the very soft silty clay and about 10 feet into the glacial till soils for stability. This results in a sheet pile retaining wall length of about 35 feet. Due to the low strength of the very soft silty clay, the top of the wall needs to be restrained. The conceptual design includes rock anchors, approximately 100 feet long spaced 10 feet apart, to restrain the upper portion of the wall.

In our opinion, it may not be practical to construct an anchored sheetpile wall at this site. Large equipment such as a crane will be required and vibrations associated with driving the sheetpiles into the dense till are likely to further destabilize the sensitive silty clay soils at this site. This could lead to instability and movement of the wall as it is installed before the rock anchors are in place to restrain the top of the wall.

2. Dry Soil Mixing

Dry soil mixing involves drilling holes down through the soft silty clay and injecting dry cement or a mixture of dry cement and lime into the soft clay as the drill progresses through the soft clay and is retracted. Cement columns installed this way may be overlapped to create larger panels. Panels are constructed perpendicular to the road to stabilize the soil beneath the road and between the road and the creek. Figure 16 is a conceptual plan layout showing the approximate extents of the dry soil mix zone and Figure 17 shows a conceptual cross section of this option.

This method has been used successfully on several sites with similar conditions. The equipment involved is about the size of a large excavator and does not require large cranes, as would be necessary for installing a sheetpile wall making access easier. Additionally, with this method, vibrations during construction are limited. For these reasons, we believe that this method is more constructable than trying to install an anchored sheetpile wall.

The spacing of the panels and the appropriate mixture of cement or cement and lime depend on the results of laboratory testing using soils from the site. We have initiated testing to provide information necessary to design the panel layout and develop the appropriate dry mix. To date we have received unconfined compressive test strength results after seven days as presented in Appendix D. The test results indicate that for a cement addition rate of 50 kg per cubic meter (about 5 percent by weight) the unconfined compressive strength of the test sample after seven days is 47.5 pounds per square inch (327.8 kPA). This corresponds to a shear strength of 3420 psf (163.9 kPA), about 10 times greater than the peak shear strength without the cement.

We completed slope stability analyses to evaluate the degree of soil improvement required for this alternative. The analysis depicted on "Dry Soil Mix Remediation Analysis," in Appendix E indicates a factor of safety of more than 1.5 for an average shear strength in the improved zone of 1200 psf (57.5 kPA). A panel layout covering 40 percent of the improvement zone would require a shear strength of 3000 psf (143.8 kPA) in the panels for an average shear strength in the improved zone of 1200 psf (57.5 KPA).

The initial test results indicate that this strength can be achieved with a cement addition rate of about 5 percent by weight. Prior to implementing this remediation method, a field test program will be required to evaluate the effectiveness of the mixing process. The field test should include mobilizing the mixing machine to the site, mixing several rows of columns, and testing them to evaluate how effective this process will be at this site. The results of the field test will be used to develop the final configuration and layout for the zone of soil improvement.

3. Flatten Creek Banks / Relocate Road

This approach to stabilizing the existing slope condition involves excavating the existing slope to a stable geometry. MMCE completed slope stability analyses for a revised, flattened slope considering the soft clay. For the analysis with the soft clay at the average peak strength of 350 psf (17.0 kPA) the slope would have to be flattened to 7 horizontal to 1 vertical for a factor of safety of 1.5. We also considered an analysis with the soft clay at the average remolded strength of 92 psf (4.4 kPA). For this analysis, a factor of safety of 1.5 is not achieved even for a slope of 10 horizontal to 1 vertical.

Based on these analyses, in our opinion, relocating the road to the south will not result in the same factor of safety as improving the soil and keeping the road in its present location.

4. Earthwork and Streambank Restoration

Whichever option is selected, we recommend that a mechanically stabilized slope wall be constructed to form the outside portion of the new road embankment as shown conceptually on Figures 15 and 17. Details of the mechanically stabilized earth (MSE) embankment should be developed during final design. Final design considerations include designing the embankment face to drain as flood waters recede and providing appropriate erosion protection.

We recommend that the road fill be sand and gravel and not crushed limestone or dolostone. Sand and gravel fill has a lower unit weight than crushed limestone or dolostone and will apply less stress to the subsoil. Details of the recommended road fill gradation should be developed in conjunction with the final design of the MSE wall.

As indicated on Figures 15 and 17, we recommend that a drain be installed on the inside face of the road excavation. The drain should include a gravel drainage stone surrounded by a geotextile as indicated on the figures.

We recommend that the zone between the reconstructed road and the creek be restored by placing riprap filled trenches and vegetation as shown conceptually on Figures 14 and 16. The stone filled trenches will protect the remediated road embankment from future scour as the creek continues to erode the outer bank.

The vegetative protection serves two purposes. It will provide erosion protection and reduce the creek flow velocity next to the bank, thereby providing scour protection to the reconstructed road. Secondly, utilizing native plant and grass species will integrate the restored area with the creek bank habitat corridor.

B. Cost Estimates

Comparative cost estimates are presented below for remediating the road. The cost estimates are based on remediating a length of 700 feet and include stabilizing the soft silty clay soil with either dry soil mixing or an anchored sheetpile wall. Also included are estimated costs for earthwork to remove the unsuitable soils from the site and reconstruct the roadbed.

The cost analyses do not include guard rails along the edge of the pavement, acquiring property (or right-of way revisions) that may be required and reconstructing the pavement.

Estimated Cost for Anchored Sheet Pile Wall

Item	**Cost, $**
Mobilization/Demobilization	100,000
Steel Sheet Piling	975,000
Rock Anchors	400,000
Total	**$1,475,000**
Cost per lineal foot	**$2,100**

Assumptions:
- Steel sheetpile wall 35 feet high by 700 feet.
- Rock anchors, 102 feet long spaced 10 feet on center, 70 total.

Estimated Cost for Dry Soil Mixing

Item	**Cost, $**
Mobilization/Demobilization	50,000
Dry Soil Mixing	1,500,000
Total	**$1,550,000**
Cost per lineal foot	**$2,200**

Assumptions:
- Dry mix panels spaced 6.5 feet center to center.
- Panels are 60 feet wide in the 250 foot collapsed area and 40 feet wide for the remaining 450 feet of the remediated area.

As indicated on the preceding tables, the estimated costs for the sheetpile wall or the dry mix stabilization are about the same. As discussed previously however, in our opinion the dry soil mixing is better suited to this application than the anchored sheetpile wall.

Each remediation option will also involve removal of unsuitable soils and rubble and reconstruction of the roadbed. We have estimated that a MSE wall approximately 400 feet long would be constructed to support the north side of the road. The following table summarizes our preliminary estimate of earthwork costs.

Estimated Cost for Earthwork and Bank Protection

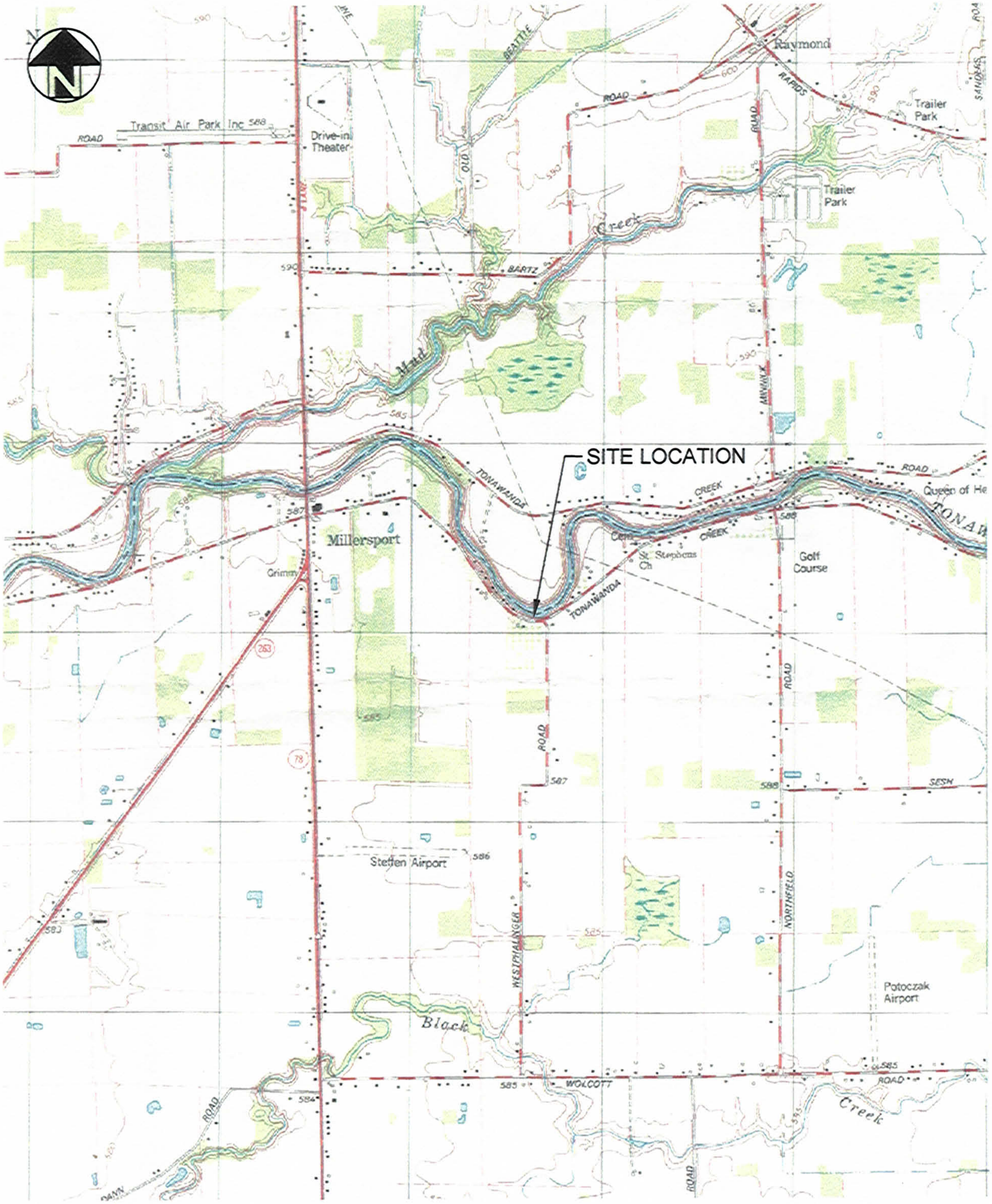
Item	**Cost, $**
Mobilization/Demobilization	20,000
Clear, grub, tree removal	45,000
Excavate and grade collapse area	175,000
MSE Wall (400 feet)	980,000
Riprap and Vegetative Protection	310,000
Total	**$1,530,000**
Cost per lineal foot	**$2,185**

Assumptions:

- Approximately 5,000 cubic yards of excavation in the collapse area.
- MSE wall would be constructed for 400 feet of the total 700 foot remediated length.

In summary, our preliminary estimate for stabilizing the soft soil and reconstructing the road bed is between about $4,000 and $4,500 per lineal foot. This does not include costs for reconstructing the pavement, property acquisitions and engineering design and construction monitoring services.

FIGURES

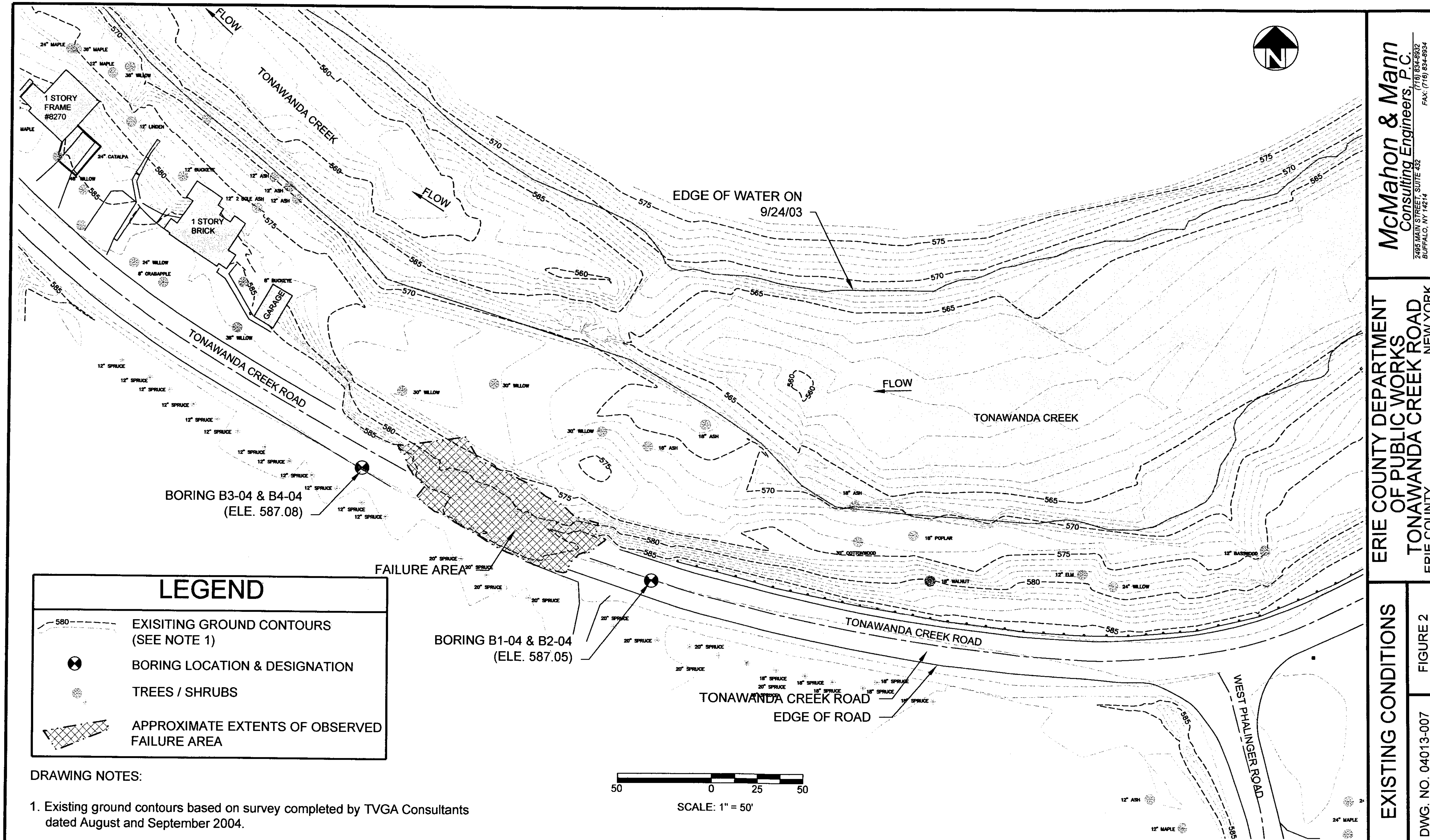


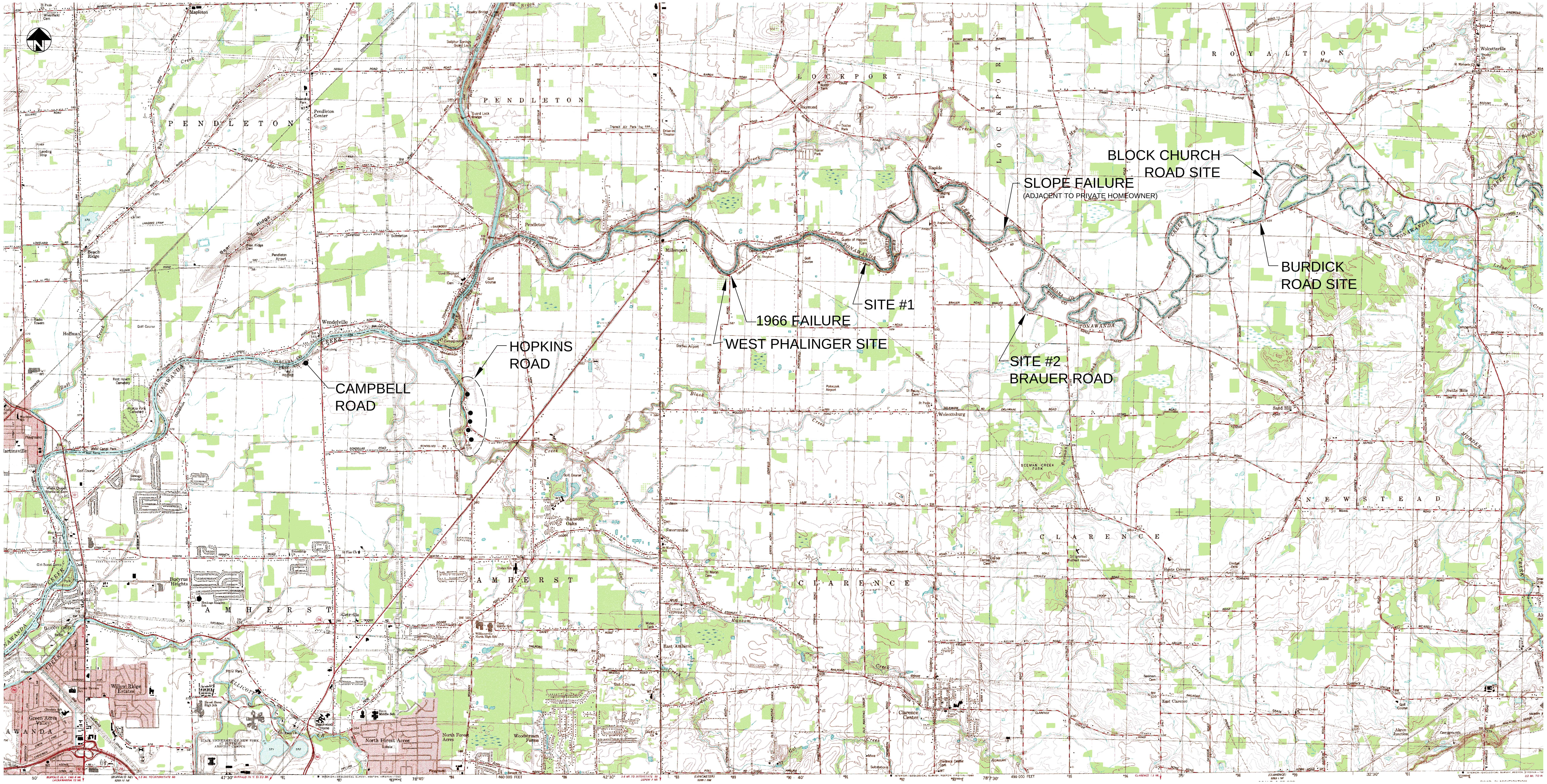
NOTE:

1. Base map created from USGS 7.5 minute quadrangle map of Clarence Center, New York, dated 1980.

APPROXIMATE SCALE: 1" = 2600'

SITE LOCATION	ERIE COUNTY DEPARTMENT OF PUBLIC WORKS TONAWANDA CREEK ROAD	McMahon & Mann Consulting Engineers, P.C.
DWG. NO. 04013-001		2495 MAIN STREET, SUITE 432 (716) 834-8932 BUFFALO, NY 14214 FAX: (716) 834-8934
FIGURE 1	ERIE COUNTY NEW YORK	


FLOW
TONAWANDA CREEK
1 STORY FRAME #8270
1 STORY BRICK
GARAGE
TONAWANDA CREEK ROAD
EDGE OF WATER ON 9/24/03
FLOW
TONAWANDA CREEK
BORING B3-04 & B4-04 (ELE. 587.08)
FAILURE AREA
BORING B1-04 & B2-04 (ELE. 587.05)
TONAWANDA CREEK ROAD
TONAWANDA CREEK ROAD
EDGE OF ROAD
WEST PHALINGER ROAD
LEGEND
EXISITING GROUND CONTOURS (SEE NOTE 1)
BORING LOCATION & DESIGNATION
TREES / SHRUBS
APPROXIMATE EXTENTS OF OBSERVED FAILURE AREA
DRAWING NOTES:
1. Existing ground contours based on survey completed by TVGA Consultants dated August and September 2004.
50 0 25 50
SCALE: 1" = 50'
McMahon & Mann
Consulting Engineers, P.C.
2495 MAIN STREET, SUITE 432
BUFFALO, NY 14214
(716) 834-8932
FAX: (716) 834-8934
ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK
EXISTING CONDITIONS
FIGURE 2
DWG. NO. 04013-007



NOTES:

1. Base map created from the following USCS quadrangle maps
   - Clarence Center, N.Y. dated 1980
   - Wolcottsville, N.Y. dated 1980
   - Tonawanda, East, N.Y. dated 1980
2. Location of selected sites are of recent or ongoing slope failure.

3000' 0 1500' 3000'

APPROXIMATE SCALE: 1" = 3000'

NOTE:
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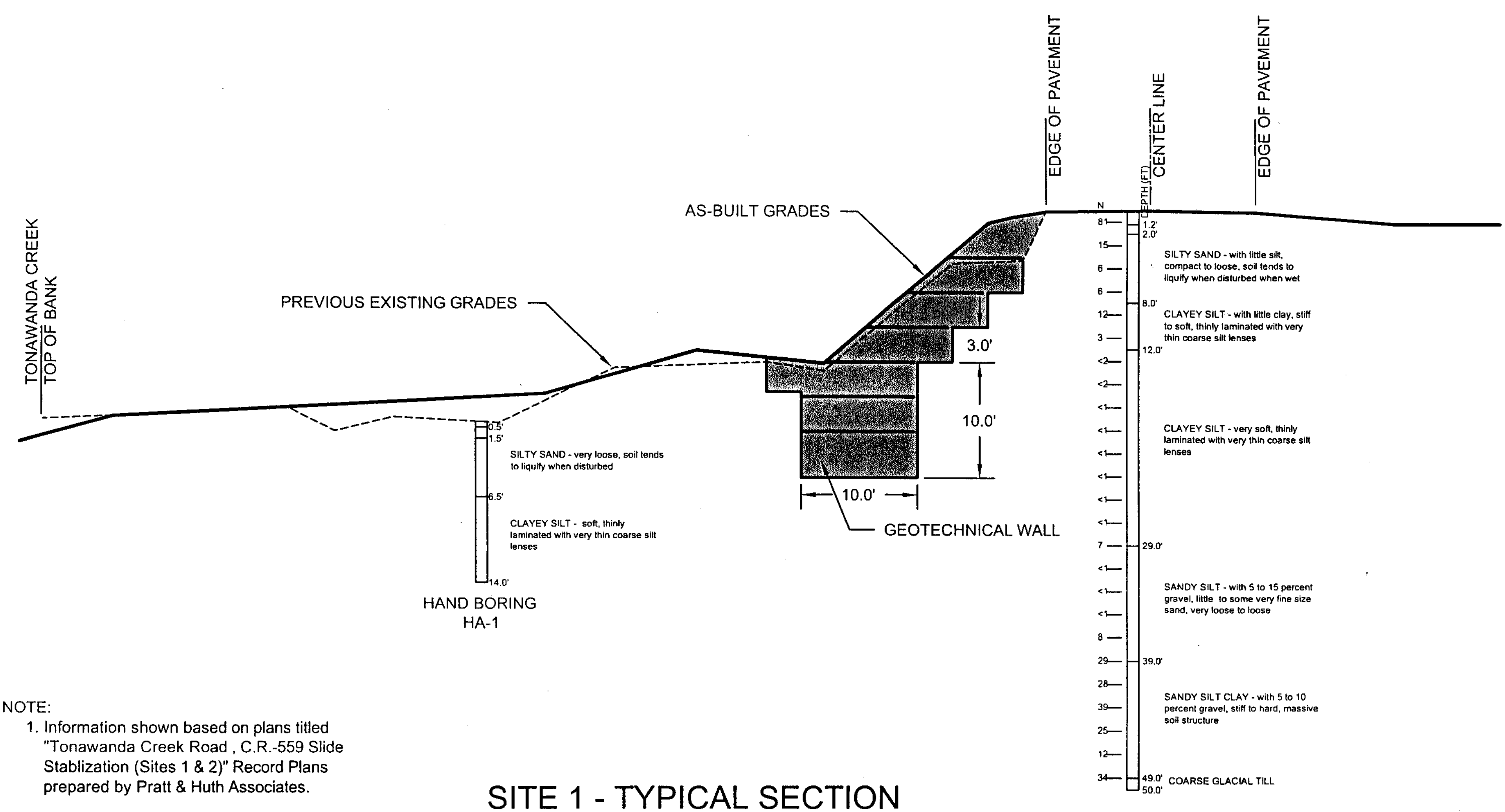
REV 1 | REV 2 | REV 3 | REV 4 | REV 5 | REV 6

ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD

NEW YORK

ERIE COUNTY

DESIGNED BY: C.R.G.
CHECKED BY: M.J.M.
SCALE: AS SHOWN
DATE: OCTOBER 2004
JOB NO. 04013
FIGURE 3
DWG. NO. 04013-004
REVISION NUMBER - 0



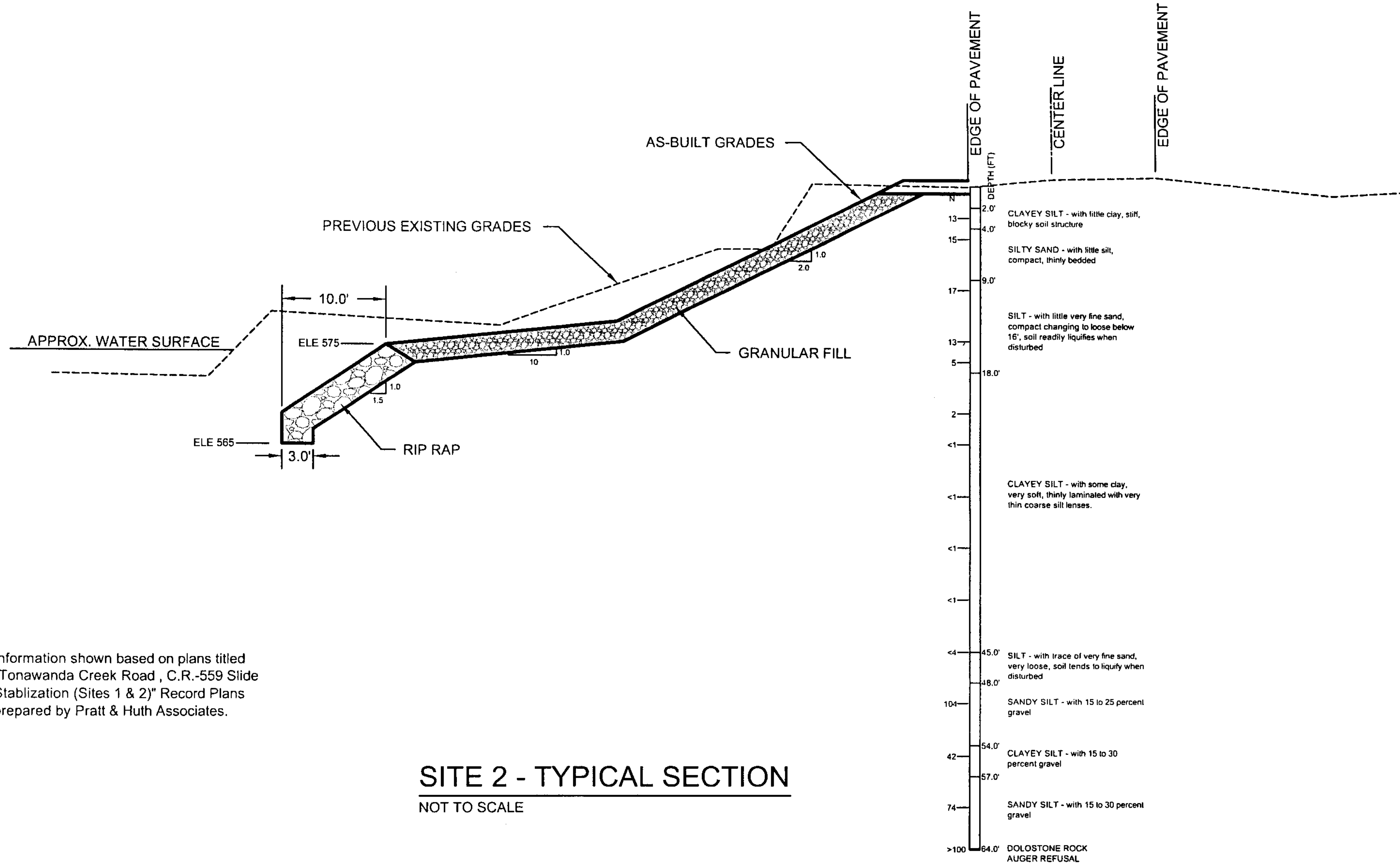
NOTE:

1. Information shown based on plans titled "Tonawanda Creek Road , C.R.-559 Slide Stablization (Sites 1 & 2)" Record Plans prepared by Pratt & Huth Associates.

SITE 1 - TYPICAL SECTION

NOT TO SCALE

McMahon & Mann Consulting Engineers, P.C. 2495 MAIN STREET, SUITE 432 BUFFALO, NY 14214 (716) 834-8932 FAX: (716) 834-8934	ERIE COUNTY DEPARTMENT OF PUBLIC WORKS TONAWANDA CREEK ROAD ERIE COUNTY NEW YORK	SITE 1 DWG. NO. 04013-010a FIGURE 4



NOTE:
1. Information shown based on plans titled "Tonawanda Creek Road , C.R.-559 Slide Stablization (Sites 1 & 2)" Record Plans prepared by Pratt & Huth Associates.

SITE 2 - TYPICAL SECTION

NOT TO SCALE

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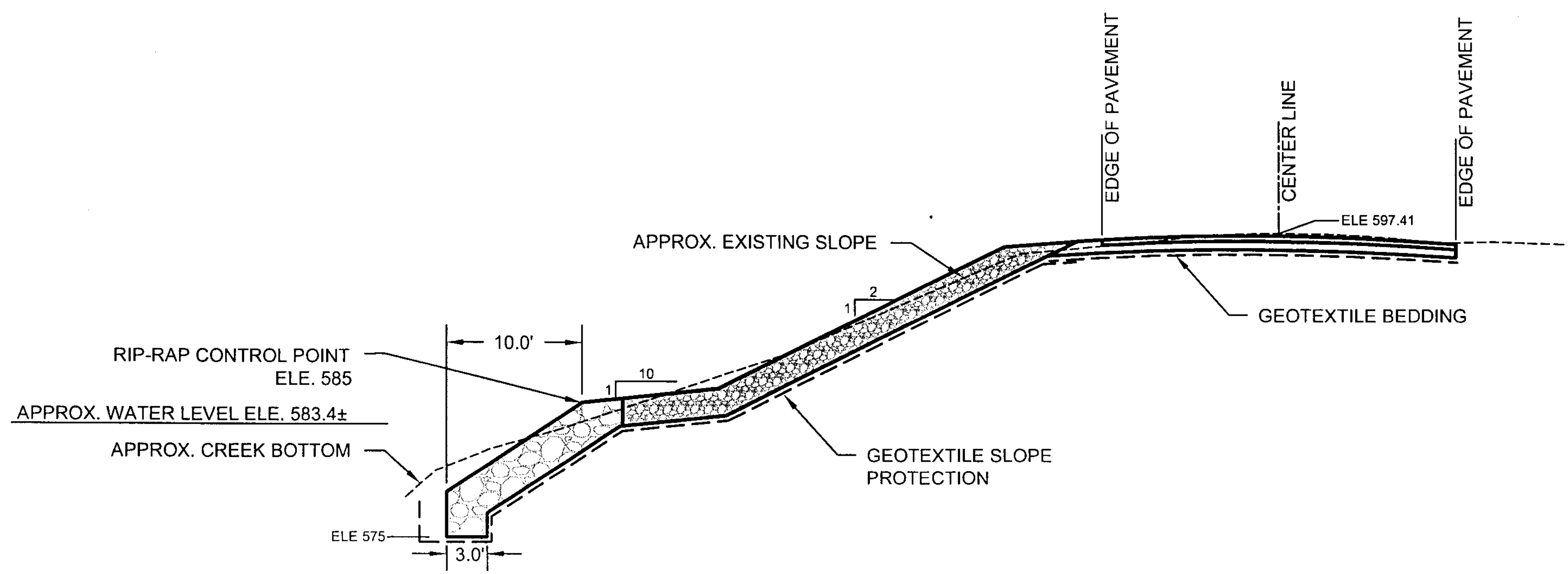
ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK

SITE 2

DWG. NO. 04013-010b

FIGURE 5



NOTE:
1. Information shown based on plans titled "Burdick Road, C.R. 258 Slide Stabilization" perpared by Abate Engineering Associates, P.C. dated June 2000.

BURDICK ROAD SITE - TYPICAL SECTION

NOT TO SCALE

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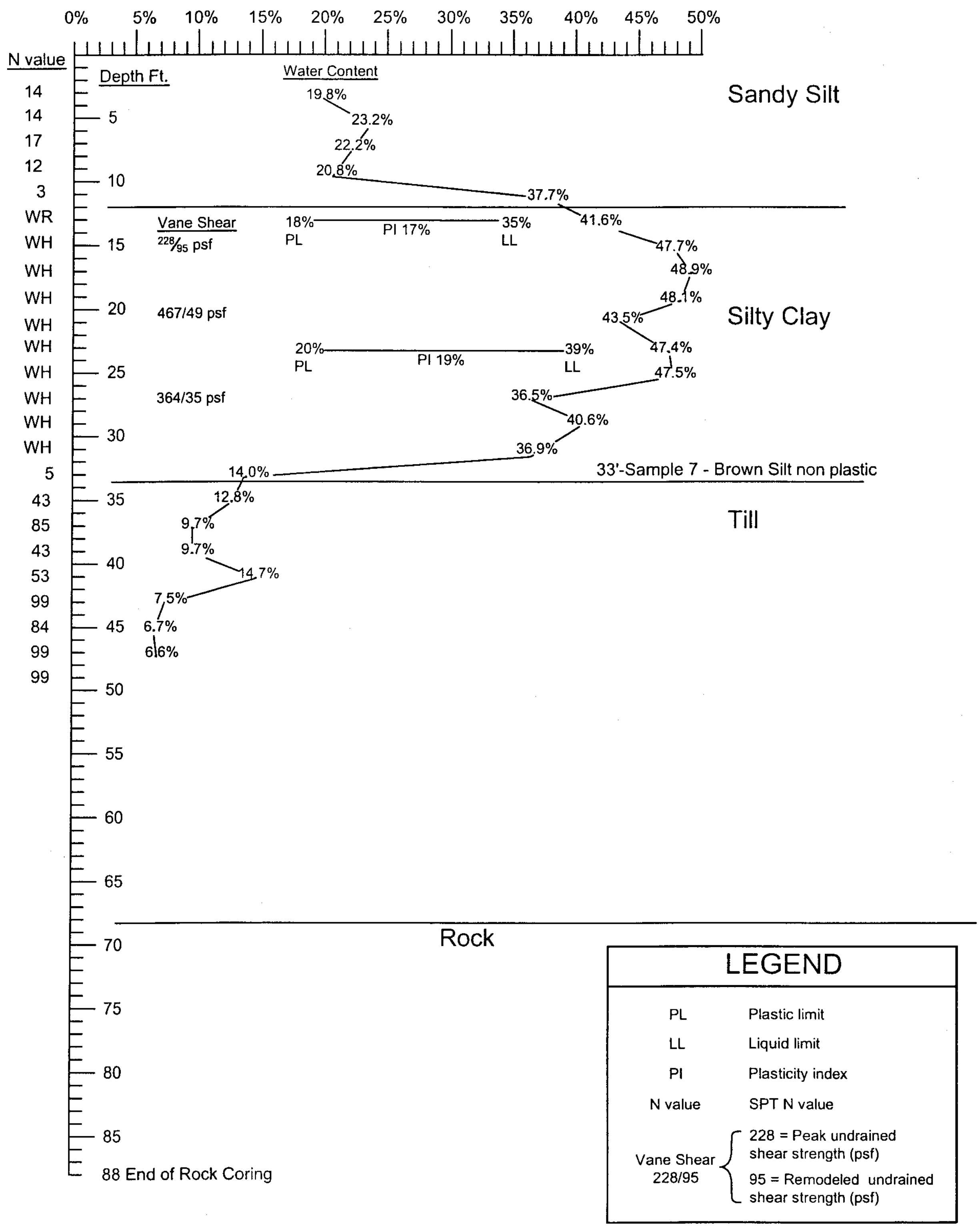
Tonawanda Creek Slope Failure

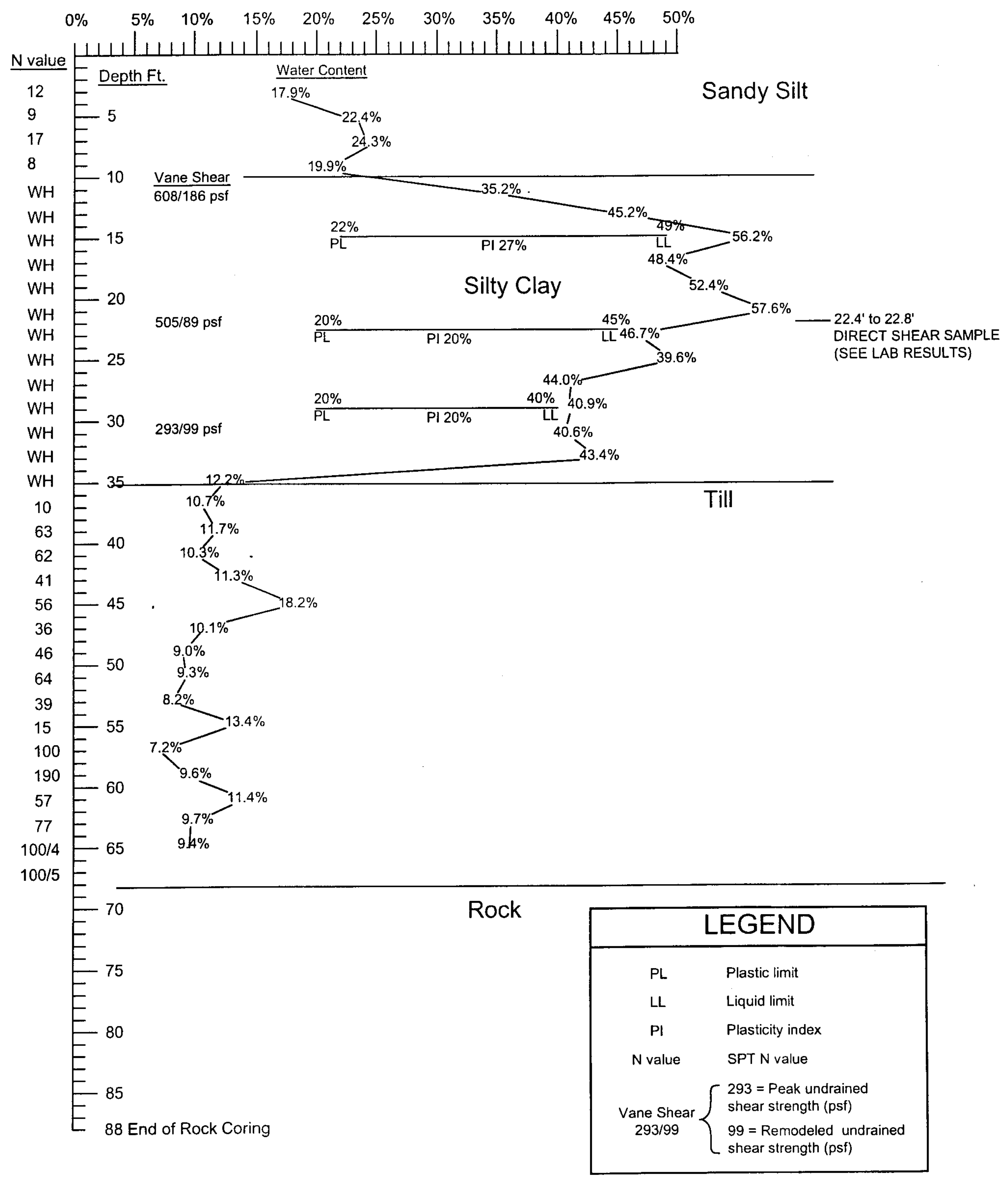
Bore B1-04 File: 04-013



BORING B1-04	ERIE COUNTY DEPARTMENT OF PUBLIC WORKS	McMahon & Mann Consulting Engineers, P.C.
DWG. NO. 04013-005a	TONAWANDA CREEK ROAD	2495 MAIN STREET, SUITE 432 BUFFALO, NY 14214 (716) 834-8932 FAX: (716) 834-8934
FIGURE 7	ERIE COUNTY NEW YORK	

Tonawanda Creek Slope Failure

Bore B3-04 File: 04-013



BORING B3-04	ERIE COUNTY DEPARTMENT OF PUBLIC WORKS	McMahon & Mann Consulting Engineers, P.C.
DWG. NO. 04013-005b	TONAWANDA CREEK ROAD	2495 MAIN STREET, SUITE 432 BUFFALO, NY 14214 (716) 834-8932 FAX: (716) 834-8934
FIGURE 8	ERIE COUNTY NEW YORK	

Figure 9 - Collapse area looking East.
Photos taken 8/2/04

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Figure 10 – Scarp, note water seeping from slope.
Photos taken 8/2/04

Figure 11 – Showing area between road and creek and soil in creek.
Photos taken 8/2/04

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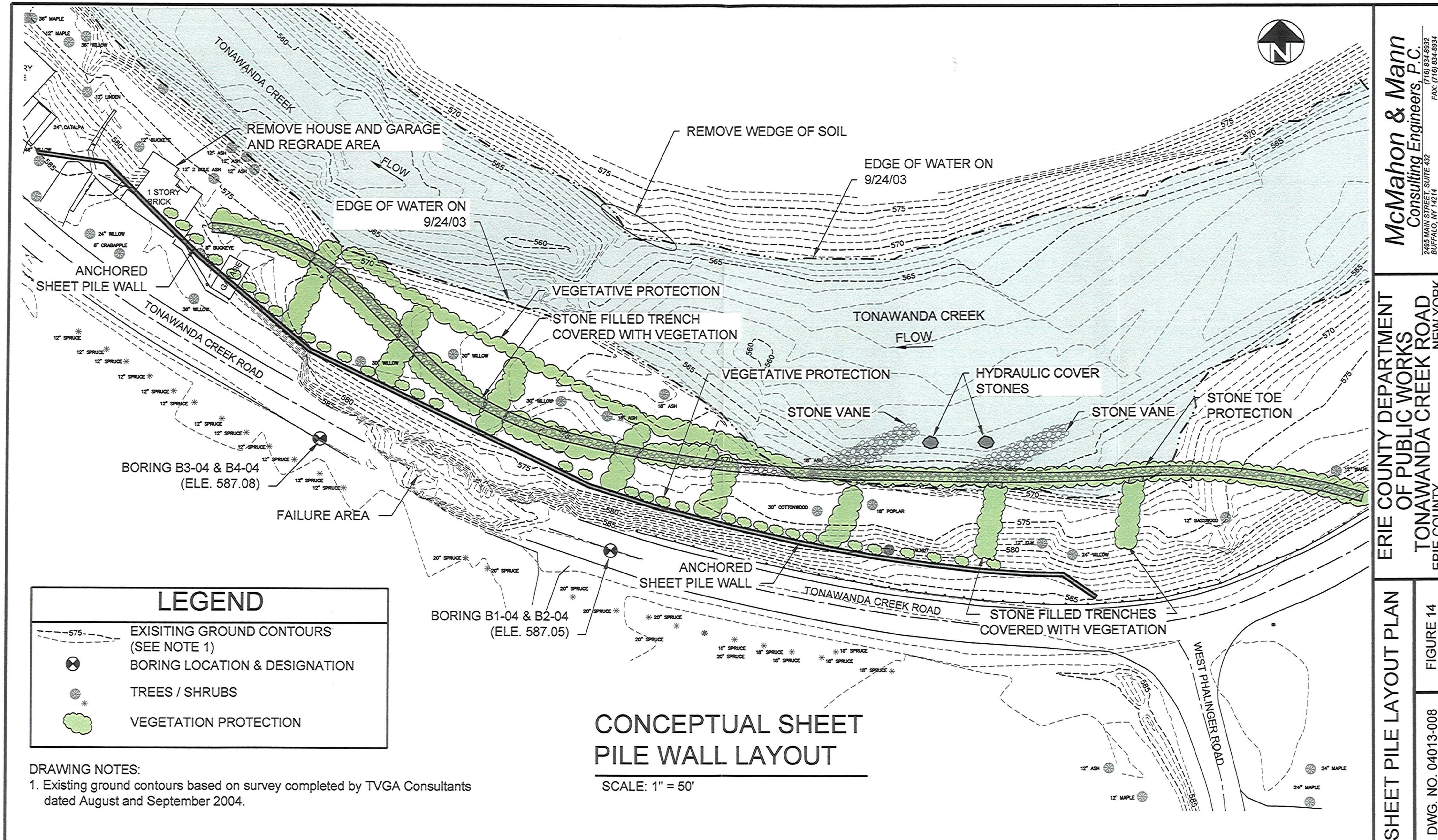
Figure 12 – Looking West at failure.
Photo taken 6/25/04

Figure 13 – Looking West at failure.
Photo taken 6/25/04



REMOVE HOUSE AND GARAGE AND REGRADE AREA
REMOVE WEDGE OF SOIL
EDGE OF WATER ON 9/24/03
EDGE OF WATER ON 9/24/03
TONAWANDA CREEK
FLOW
1 STORY BRICK
ANCHORED SHEET PILE WALL
VEGETATIVE PROTECTION
STONE FILLED TRENCH COVERED WITH VEGETATION
TONAWANDA CREEK
FLOW
VEGETATIVE PROTECTION
HYDRAULIC COVER STONES
STONE VANE
STONE VANE
STONE TOE PROTECTION
TONAWANDA CREEK ROAD
BORING B3-04 & B4-04 (ELE. 587.08)
FAILURE AREA
ANCHORED SHEET PILE WALL
TONAWANDA CREEK ROAD
STONE FILLED TRENCHES COVERED WITH VEGETATION
BORING B1-04 & B2-04 (ELE. 587.05)
WEST PHALINGER ROAD
LEGEND
EXISITING GROUND CONTOURS (SEE NOTE 1)
BORING LOCATION & DESIGNATION
TREES / SHRUBS
VEGETATION PROTECTION
CONCEPTUAL SHEET PILE WALL LAYOUT
SCALE: 1" = 50'
DRAWING NOTES:
1. Existing ground contours based on survey completed by TVGA Consultants dated August and September 2004.
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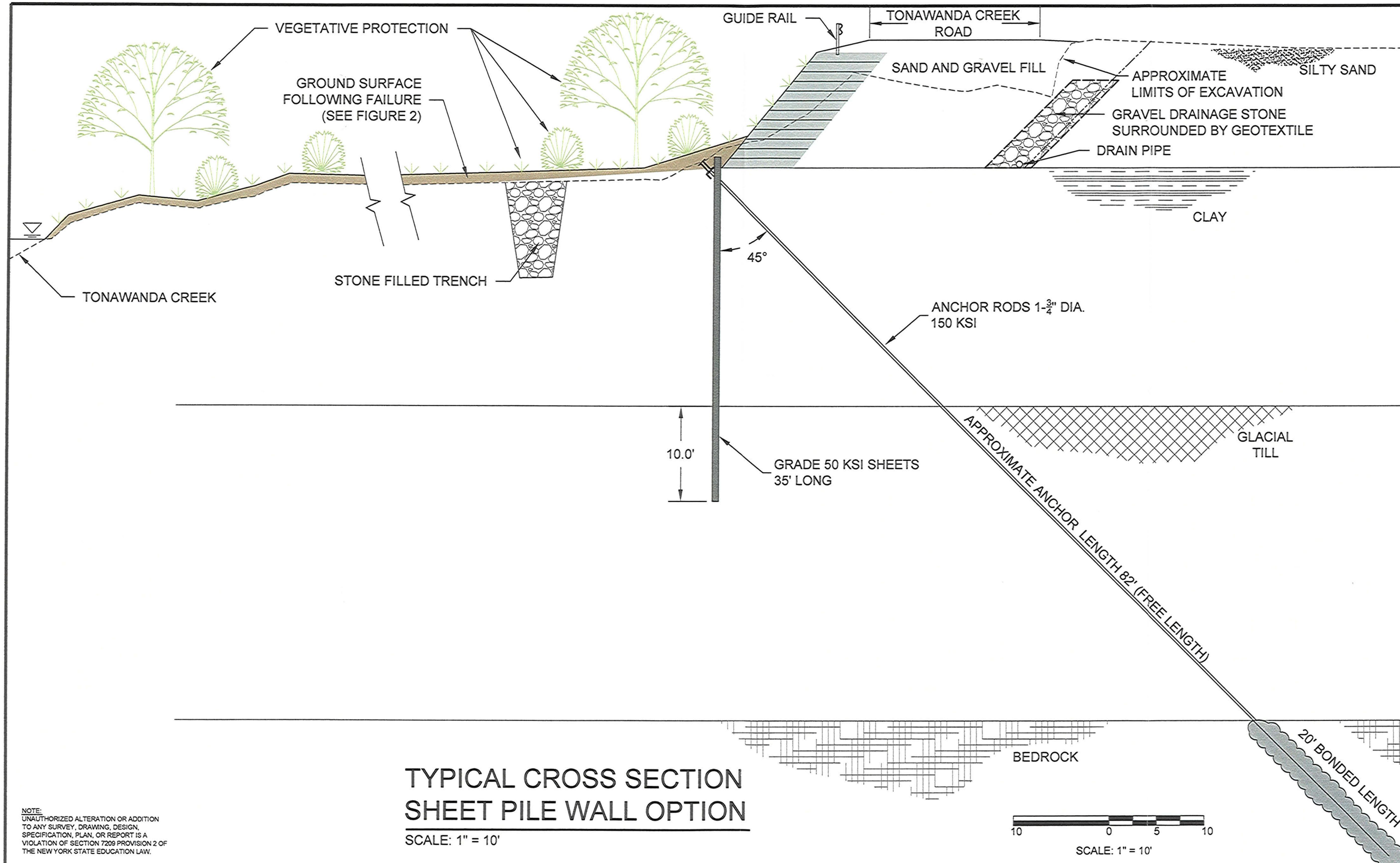
ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK
SHEET PILE LAYOUT PLAN
FIGURE 14
DWG. NO. 04013-008



TYPICAL CROSS SECTION
SHEET PILE WALL OPTION

SCALE: 1" = 10'

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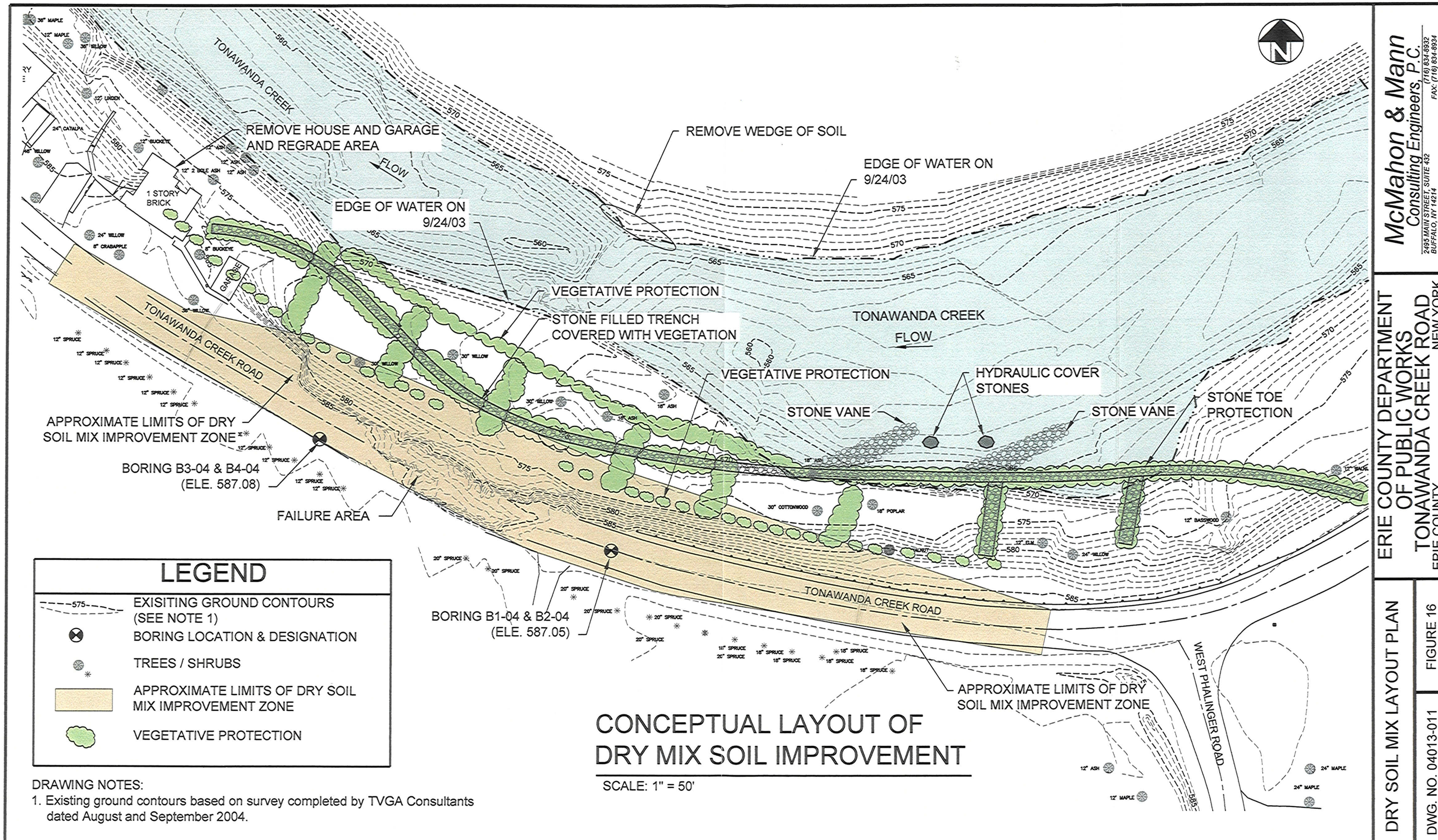
ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK

SHEET PILE WALL SECTION

FIGURE 15

DWG. NO. 04013-006B



TONAWANDA CREEK
REMOVE HOUSE AND GARAGE AND REGRADE AREA
FLOW
1 STORY BRICK
GARAGE
EDGE OF WATER ON 9/24/03
REMOVE WEDGE OF SOIL
EDGE OF WATER ON 9/24/03
VEGETATIVE PROTECTION
STONE FILLED TRENCH COVERED WITH VEGETATION
TONAWANDA CREEK
FLOW
VEGETATIVE PROTECTION
HYDRAULIC COVER STONES
STONE VANE
STONE VANE
STONE TOE PROTECTION
TONAWANDA CREEK ROAD
APPROXIMATE LIMITS OF DRY SOIL MIX IMPROVEMENT ZONE
BORING B3-04 & B4-04 (ELE. 587.08)
FAILURE AREA
BORING B1-04 & B2-04 (ELE. 587.05)
TONAWANDA CREEK ROAD
APPROXIMATE LIMITS OF DRY SOIL MIX IMPROVEMENT ZONE
WEST PHALINGER ROAD
LEGEND
EXISITING GROUND CONTOURS (SEE NOTE 1)
BORING LOCATION & DESIGNATION
TREES / SHRUBS
APPROXIMATE LIMITS OF DRY SOIL MIX IMPROVEMENT ZONE
VEGETATIVE PROTECTION
CONCEPTUAL LAYOUT OF DRY MIX SOIL IMPROVEMENT
SCALE: 1" = 50'
DRAWING NOTES:
1. Existing ground contours based on survey completed by TVGA Consultants dated August and September 2004.
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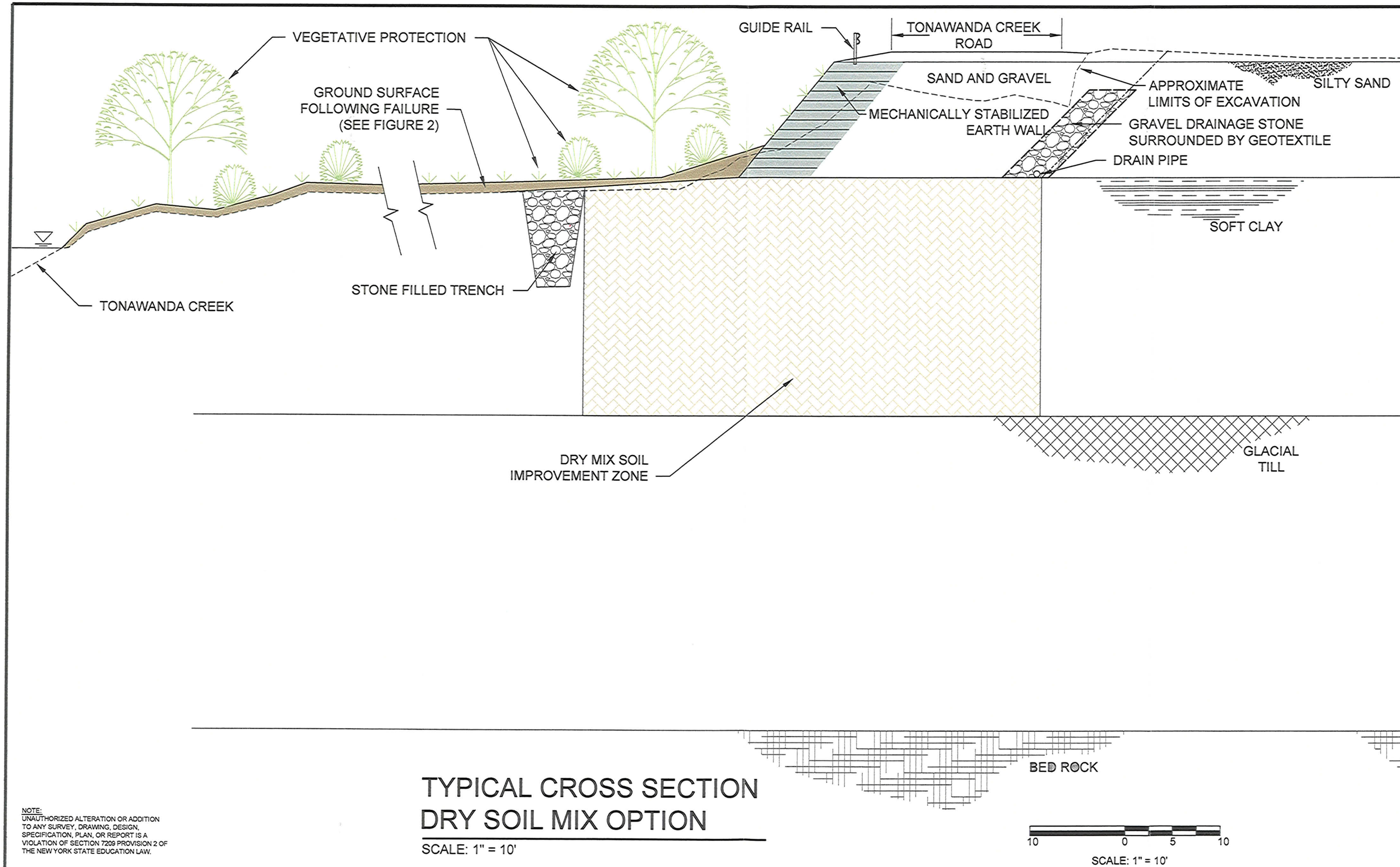
ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK
DRY SOIL MIX LAYOUT PLAN
FIGURE 16
DWG. NO. 04013-011



TYPICAL CROSS SECTION
DRY SOIL MIX OPTION

SCALE: 1" = 10'

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ERIE COUNTY DEPARTMENT OF PUBLIC WORKS
TONAWANDA CREEK ROAD
ERIE COUNTY NEW YORK

DRY MIX SECTION

FIGURE 17

DWG. NO. 04013-006C

APPENDIX A

SUMMARY OF SUBSURFACE EXPLORATIONS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

McMahon & Mann
Consulting Engineers, P.C.

APPENDIX A

SUMMARY OF SUBSURFACE EXPLORATIONS
TONAWAND CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

I. TEST BORINGS

In August 2004 Earth Dimensions, Inc. (EDI) used a truck mounted drill rig to make four test borings at this site. These borings are designated Borings B1-04, B2-04, B3-04 and B4-04. The location of the test borings is shown on Figure 2.

The borings were made using hollow stem augers. Soil samples were collected from below the bottom of the augers as they were advanced. Generally, sampling began at the ground surface and continued at 2 foot (0.6 meter) intervals.

Soil samples were collected using a 1-3/8 inch (34.9 millimeter [mm]) inside diameter, 24 inch (610 mm) long split spoon sampler, in general accordance with ASTM method D 1586. Samples were obtained by driving the sampler into the ground with a 140 pound (63.5 kilogram) hammer falling 30 inches (762 mm). The sampler was driven 24 inches (610 mm), the soil sample was removed from the sampler and a description of the sample was recorded on the boring log.

The number of hammer blows required to drive the sampler 6 inches (152 mm) was recorded. The sum of the number of blows required to advance the sampler in the second and third 6-inch (152 mm) interval is known as the Standard Penetration Test (SPT) N-value.

The SPT N-values are correlated to the consistency of the clayey silt soil, as shown below:

Consistency	N-value
Very Soft	<2
Soft	2-4
Medium	4-8
Stiff	8-15
Very Stiff	15-30
Hard	>30

The SPT N-values are also correlated to the density of granular soils as shown below:

Density	N-value
Very Loose	0-4
Loose	4-10
Medium	10-30
Dense	30-50
Very Dense	>50

An EDI soil scientist monitored the test borings, observed the soil samples and prepared a log of the conditions encountered. The logs for the test borings follow.

EDI collected thin-walled soil samples (nominal 3-inch diameter) generally following the procedures described in ASTM method D 1587. These samples were collected from borings B2-04 and B4-04 at the depths indicated on the attached logs. The Shelby tube samples were collected from below the Vane Shear Test locations and provide samples for measurement of unit weight and shear strength. The Shelby tube samples were also mixed with dry cement to provide data relative to the potential strength gain for this remedial option. The vane shear test results are presented in Appendix B and soil laboratory test data is in Appendix D.

EDI measured the depth to water in the test borings upon completion and recorded those measurements on the test boring logs. These measurements may not represent static ground water depths as sufficient time may not have elapsed for ground water levels to have stabilized during drilling.

EDI collected 20 feet of rock core from Boring B1-04 and 10 feet from B3-04. Rock was cored using an HQ size core barrel, which yields an approximate 2-inch diameter core sample. The rock type and the condition of the core are recorded on the logs. The percentage core recovery and the rock quality designation (RQD) for each core run are also recorded on the log. The percentage recovery is the length of rock core recovered divided by the length of the core run, in percent. The RQD is a measure of the rock quality and is defined as the sum of the core pieces that are 4 inches or greater in length divided by the length of the core run.

II. PIEZOMETER

A piezometer, designated as OW 1-04, was installed in the vicinity of boring locations B3-04 and B4-04. The piezometer is constructed of 2-inch (50.8 mm) diameter PVC well screen and riser pipe and allows access for measurement of groundwater levels with time. Refer to the attached log for details of the installation. Groundwater level measurements are presented in Appendix C.

III. INCLINOMETERS

EDI installed inclinometer casing having an approximate diameter of 2.75 inches, in Boring B1-04. The casing was supplied by the Slope Indicator Co. of Seattle, Washington. The inclinometer tip is set below the top of rock as indicated on the log for B1-04. Using an inclinometer sensor, MMCE measured the verticality of the casing at various times after its installation. These measurements are presented in Appendix C and provide data relative to lateral movement of the ground with depth.

EARTH DIMENSIONS, INC.

Soil and Hydrogeologic Investigations • Wetland Delineations
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(716) 655-1717 • FAX (716) 655-2915

24G04 HOLE NO. Bore Hole 01-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/02/04 COMPLETED 08/05/04

DEPTH IN FT / BLOWS ON SAMPLER

SN / REC	0/6	6/12	12/18	18/24	N
1 / 8	20	14			
2 / 21	9	8	6	7	14
3 / 23	5	6	8	9	14
4 / 24	7	8	9	12	17
5 / 20	5	6	6	7	12
6 / 24	2	2	1	1	3
7 / 24	WR	WR	WH	WH	<1
8 / 24	WR	WH	WH	WH	<1
9 / 22	WR	WH	WH	WH	<1
10 / 24	WR	WR	WH	WH	<1

Depth markers: 5, 10, 15, 20

DESCRIPTION AND CLASSIFICATION

Gray asphalt pavement 0.9

Moist gray very gravelly (SAND) fill with 40 to 60% mostly crushed stone, very fine to very coarse size sand, trace silt, compact, loose when disturbed, single grain, (SW), (GW). 2.0

Moist faintly mottled brown (CLAYEY-SILT) with some clay, very stiff, blocky soil structure, (CL).

clear transition to 2.7

Extremely moist faintly mottled brown (SILTY-SAND) with mostly very fine to fine size sand, little silt, compact, thinly bedded, (SM).

clear transition to 6.8

Extremely moist to wet brownish gray (SILT), trace to very fine size sand, compact, slight liquification when disturbed, thinly bedded, (ML).

grades downward to 8.5

Extremely moist to wet (CLAYEY-SILT) with little clay, stiff, thinly laminated with very thin coarse silt lenses, (ML-CL).

grades downwa to 10.0

Extremely moist, wet below 0 feet, (SILTY-CLAY), soft, very soft below 11.0 feet, thinly laminated with very thin coarse silt lenses, (CL-CH).

clear transition to 19.8

See next sheet.

WELL

3.34 O.D. INCLINOMETER CASING

CEMENT BENTONITE GROUT

WATER TABLE AND REMARKS

Gray asphalt pavement to 0.9 feet over mostly crushed stone fill to 2.0 feet over clayey slack water sediment to 2.7 feet over water sorted and deposited sand with little silt to 6.8 feet over silty slack water sediment with trace sand to 8.5 feet over clayey lake sediment to 31.6 feet over silty glacial drift with little sand, gravel and clay to 34.0 feet over loamy glacial till to 40.3 feet over clayey slack water sediment to 41.0 feet over mostly loamy glacial till to 68.4 feet over dolomite bedrock to end of coring.

(1) Flush mount roadway box installed in concrete.

WR - Sampler penetration with weight of rods.

WH - Sampler penetration with weight of rods and hammer.

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cis)

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24G04 HOLE NO. Bore Hole 01-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/02/04 COMPLETED 08/05/04

DEPTH IN FT / BLOWS ON SAMPLER

DEPTH IN FT	SN / REC	0/6	6/12	12/18	18/24	N
	11 / 24	WR	WH	WH	1	<1
	12 / 24	WR	WH	WH	WH	<1
25	13 / 24	WR	WH	1	2	<2
	14 / 24	WR	WR	WH	WH	<2
30	15 / 20	WR	WR	WH	WH	<1
	16 / 20	WR	WH	WH	2	<1
	17 / 14	2	3	2	5	5
35	18 / 15	8	17	26	18	43
	19 / 20	29	39	46	30	85
40	20 / 24	18	18	25	25	43

DESCRIPTION AND CLASSIFICATION

Wet alternating reddish brown and gray (SILTY-CLAY), very soft, weakly thinly laminated with very thin coarse silt lenses, (CL-CH).

clear transition to 31.6

Wet grayish brown (SAND-SILT-CLAY) with 5 to 10% gravel, little sand and clay, massive soil structure, (ML-CL).

34.0

Extremely moist grayish brown gravelly (SAND-SILT-CLAY) with 15 to 40% mostly subangular gravel, little sand and clay, very stiff, massive soil structure, (SC).

35.0

Wet grayish brown gravelly (SILTY-SAND) with 15 to 25% gravel, little silt, dense, weakly stratified to massive soil structure, (SM).

grades downward to 36.0

See next sheet.

WELL

3.34 O.D. INCLINOMETER CASING

CEMENT BENTONITE GROUT

WATER TABLE AND REMARKS

WR – Sampler penetration with weight of rods.

WH – Sampler penetration with weight of rods and hammer.

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cis)

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24G04 HOLE NO. Bore Hole 01-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/02/04 COMPLETED 08/05/04

DEPTH IN FT / BLOWS ON SAMPLER

SN	REC	0/6	6/12	12/18	18/24	N
21	21	7	13	40	55	53
22	24	21	43	56	73	99
23	24	31	39	45	60	84
24	24	36	44	55	70	99
25	22	39	41	58	51	99
26	24	21	48	53	88	101
27	22	99	57	37	61	94
28	3	68	100/4			
		Core	Run	#1		

Depth markers: 45, 50, 55, 60

DESCRIPTION AND CLASSIFICATION

Extremely moist grayish brown gravelly (SILTY-SAND) with 15 to 25% mostly subangular gravel, little silt, very dense with slight brittle consistence, massive soil structure, (SM).
grades downward to — 37.5

Extremely moist to moist grayish brown gravelly (SAND-SILT-CLAY) with 5 to 15% mostly subangular gravel, little sand and clay, dense with slight brittle consistence, massive soil structure, (SM). — 40.3

Extremely moist grayish brown (CLAYEY-SILT) with little clay, very stiff, thinly laminated, (ML-CL). — 41.0

Extremely moist grayish brown (SAND-SILT-CLAY) with 5 to 15% mostly subangular gravel, little sand and clay, hard, massive soil structure, (ML-CL).
grades downward to — 42.0

Moist brownish gray gravelly (SANDY-SILT) with 15 to 40% mostly subangular gravel, little sand, very dense with brittle consistence, massive soil structure, (SM). — 54.0

Boulder. — 56.5

WELL: 3.34 O.D. INCLINOMETER CASING; CEMENT BENTONITE GROUT

WATER TABLE AND REMARKS

Note: Drilled with 3 7/8" roller bit 54.0 to 56.0 feet. Cored 56.0 to 66.0 feet (recovered boulder core, gravel, till and clayey sediment).

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cis)

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24G04 HOLE NO. Bore Hole 01-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/02/04 COMPLETED 08/05/04

DEPTH IN FT / BLOWS ON SAMPLER

Depth	SN / REC	0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WELL	WATER TABLE AND REMARKS
		Core	Run	#1					3.34 O.D. INCLINOMETER CASING; CEMENT BENTONITE GROUT	Note: Drilled with 3 7/8" roller bit to 66.0 feet and sampled, continued with 3 7/8" roller bit into bedrock at split spoon 68.4 feet to 69.0 feet. Cored rock with NQ size double tube core barrell and diamond bit.
65										
66.0	29	56						Moist grayish brown gravelly (SANDY-SILT) with 15 to 40% mostly subangular gravel, little sand, very dense with slight brittle consistence, massive soil structure, (SM).		
	14		72							
				100/4						
68.4								Gray dolomite bedrock, effervesses only when etched, moderately hard, easily etched with knife, moderately fractured horizontally with gypsum deposits, several low angle fractures, dense to very fine crystalline.		
70									BENTONITE SEAL	← 70.0
		Core	Run	#2						
75										
80		Core	Run	#3						

Run #	Depth (ft)	Length (ft)	Rec (ft)	Rec %	RQD %
1	56.0 to 64.0	8.0	N/A	N/A	N/A
2	69.0 to 79.0	10.0	9.8	98%	63%
3	79.0 to 89.0	10.0	9.9	99%	71%

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cjs)

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(716) 655-1717 • FAX (716) 655-2915

24G04 HOLE NO. Bore Hole 01-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/02/04 COMPLETED 08/05/04

DEPTH IN FT	SN / REC	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WELL	WATER TABLE AND REMARKS
		Core	Run	#3				Gray dolomite bedrock, effervesses only when etched, moderately hard, easily etched with knife, moderately fractured horizontally with gypsum deposits, several low angle fractures, dense to very fine crystalline. 85.5	BENTONITE SEAL	
85								Brownish gray dolomite bedrock, effervesses only when etched, moderately hard, easily etched with knife, non-fractured, very fine crystal to crystalline, porous. 89.0		← 89.0
90								Boring completed at 89.0 feet.		
95										
100										

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cjs) SHEET 5 OF 5

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24G04 HOLE NO. Bore Hole 02-04 SURF. ELEVATION 587.05

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/05/04 COMPLETED 08/06/04

DEPTH IN FT / BLOWS ON SAMPLER

DEPTH	SN / REC	0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WATER TABLE AND REMARKS
0–11.0								Advanced bore hole with 4 1/4" I.D. hollow stem auger casing without split spoon sampling to 11.0 feet.	
5									
10								11.0	
	1 / 24	1	1	1	1	2		Wet brownish gray (SILTY-CLAY), very soft, weakly thinly laminated with very thin coarse silt lenses, (CL-CH). 13.0	
	VS-1								VS-1 Vane shear testing conducted between 13.2 and 14.8 foot depth
	VS-1								
	VS-1								
15	VS-1								
	ST-1								ST-1 Shelby tube soil sample taken between 15.0 to 17.0 foot depths REC = 24/24
	ST-1								
	ST-1								
	ST-1								
	ST-1								
	VS-2								VS-2 Vane shear testing conducted between 19.0 and 20.5 foot depth
20	VS-2								

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

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24G04 HOLE NO. Bore Hole 02-04 SURF. ELEVATION 587.05

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Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/05/04 COMPLETED 08/06/04

DEPTH IN FT	SN	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WATER TABLE AND REMARKS
	VS-2								
	ST-2								ST-2 Shelby tube soil sample taken between 21.0 to 23.0 foot depths REC 14/24
	ST-2								
	ST-2								
	ST-2								Recovered 14 inches.
25									
	VS-3								VS-3 Vane shear testing conducted between 26.0 to 27.5 foot depths
	VS-3								
	VS-3								
	ST-3								ST-3 Shelby tube soil sample taken between 28.0 to 30.0 foot depths, REC 24/24
	ST-3								
	ST-3							30.0	
	ST-3								
30								Boring completed at 30.0 feet.	Recovered 24 inches.
35									
40									

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

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24G04 HOLE NO. Bore Hole 03-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/09/04 COMPLETED 08/11/04

DEPTH IN FT	SN	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N
	1	16				
	7		5			
	2	4				
	22		5			12
				7		
					6	
	3	3				
5	24		3			9
				6		
					5	
	4	7				
	22		7			17
				10		
					10	
	5	6				
	24		4			8
				4		
10					4	
	6	1				
	24		WH			<2
				1		
					2	
	7	WH				
	24		WH			<1
				WH		
					1	
	8	WH				
15	24		WH			<1
				WH		
					WH	
	9	WR				
	24		WR			<1
				WH		
					WH	
	10	WR				
	24		WR			<1
				WR		
20					WH	

DESCRIPTION AND CLASSIFICATION

Gray asphalt pavement. 1.0

Moist gray very gravelly (SAND) fill with 40 to 60% gravel, very fine to very coarse size sand, trace silt, compact, loose when disturbed, single grain, (SW), (GW). 2.0

Extremely moist distinctly mottled brown (SAND-SILT-CLAY) with little clay and very fine to fine size sand, stiff, blocky soil structure, (ML-CL).
grades downward to 2.5

Moist distinctly mottled brown (SAND) with mostly very fine to fine size sand, trace silt, compact, weakly bedded, (SP).
grades downward to 5.0

Wet distinctly mottled brown (SAND) with mostly very fine to fine size sand, loose, liquifies when disturbed, weakly bedded, (SP). 6.0

Wet gray (SANDY-SILT) with trace to little mostly very fine size sand, compact, liquifies when disturbed, thinly bedded, (ML).
grades downward to 9.0

Extremely moist alternating reddish brown and grayish brown (SILTY-CLAY), firm, thinly laminated with very thin coarse silt lenses, (CL).
grades downward to 10.0

Wet grayish brown (SILTY-CLAY), very soft, weakly thinly laminated with very thin coarse silt lenses, (CL-CH).

WATER TABLE AND REMARKS

Gray asphalt pavement to 1.0 feet over mostly sand and gravel fill to 2.0 feet over silty slack water sediment with little clay and sand to 2.5 feet over water sorted and deposited sand to 6.0 feet over coarse silty slack water sediment with trace to little sand to 9.0 feet over clayey lake sediment to 33.8 feet over silty glacial drift with little gravel, sand and clay to 37.0 feet over loamy glacial till to 42.0 feet over silty slack water sediment with little sand and clay to 45.0 feet over coarse silty slack water sediment with little sand to 46.0 feet over loamy glacial till to 52.5 feet over clayey slack water sediment to 53.0 feet over water sorted and deposited sand to 54.0 feet over water sorted and deposited sand with little gravel to 56.0 feet over water sorted and deposited sand with little to some gravel and occasional cobble and boulder to 61.0 feet over loamy glacial till to 67.8 feet over dolomite bedrock to end of coring.

WH – Sampler penetration with weight of rods.

WR – Sampler penetration with weight of rods and hammer.

N=NUMBER OF BLOWS TO DRIVE 2" SPOON 12" WITH 140 lb. WT. FALLING 30" PER BLOW

LOGGED BY Brian Bartron, Geologist. (cjs)

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24G04 HOLE NO. Bore Hole 03-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION –
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/09/04 COMPLETED 08/11/04

DEPTH IN FT / BLOWS ON SAMPLER

Depth	SN	0/6	6/12	12/18	18/24	N
	11 / 24	WR	WH	WH	WH	<1
	12 / 24	WH	1/12		1	<1
25	13 / 24	WR	WR	WH	WH	<1
	14 / 24	WR	WH	WH	1	<1
	15 / 24	WR	WH	WH	WH	<1
30	16 / 24	WR	WR	WH	1	<1
	17 / 24	WR	WR	WH	WH	<1
35	18 / 14	WR	WR	WR	WH	<1
	19 / 24	WH	2	8	20	10
40	20 / 24	8	24	39	32	63

DESCRIPTION AND CLASSIFICATION

Wet grayish brown (SILTY-CLAY), very soft, weakly thinly laminated with very thin coarse silt lenses, (CL-CH).
grades downward to 20.5

Wet alternating reddish brown and grayish brown (SILTY-CLAY), very soft, weakly thinly laminated with very thin coarse silt lenses, (CL-CH).
clear transition to 33.8

Wet reddish brown gravelly (SAND-SILT-CLAY) with 15 to 20% gravel, little sand and clay, very soft, massive soil structure, (ML-CL).
grades downward to 37.0

Extremely moist grayish brown gravelly (SANDY-SILT) with 15 to 20% mostly subangular gravel, little sand, compact, massive soil structure, (SM).
grades downward to 38.5

See next sheet.

WATER TABLE AND REMARKS

WR – Sampler penetration with weight of rods and hammer.

WH – Sampler penetration with weight of rods.

N=NUMBER OF BLOWS TO DRIVE 2" SPOON 12" WITH 140 lb. WT. FALLING 30" PER BLOW

LOGGED BY Brian Bartron, Geologist. (cjs)

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PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION –
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/09/04 COMPLETED 08/11/04

DEPTH IN FT / BLOWS ON SAMPLER

DEPTH	SN	0/6	6/12	12/18	18/24	N
	21 / 24	15	30	33	41	63
	22 / 24	18	18	23	28	41
45	23 / 24	17	25	31	29	56
	24 / 22	12	17	19	22	36
	25 / 16	17	21	25	19	46
50	26 / 18	22	26	31	31	57
	27 / 18	14	18	21	28	39
55	28 / 16	7	7	8	15	15
	29 / 6	22	38	62	51	100
60	30 / 8	43	105	85	39	190

DESCRIPTION AND CLASSIFICATION	WATER TABLE AND REMARKS
Moist grayish brown gravelly (SANDY-SILT) with 15 to 40% mostly subangular gravel, little sand, very dense, massive soil structure, (SM).	
grades downward to 42.0	
Moist brown (SAND-SILT-CLAY) with 3 to 7% gravel, little sand and clay, dense, massive soil structure, (ML-CL).	
clear transition to 45.0	
Wet brown (SANDY-SILT) with little mostly very fine size sand, very dense, liquifies when disturbed, thinly bedded, (ML).	
clear transition to 46.0	
Moist brown (SANDY-SILT) with 15 to 40% mostly subangular gravel, little sand, dense, massive soil structure, (SM).	
clear transition to 52.5	
Moist reddish brown (SILTY-CLAY), hard, thinly laminated with very thin coarse silt lenses, (CL).	
clear transition to 53.0	
Wet gray (SAND) with very fine to very coarse size sand, dense, liquifies when disturbed, stratified, (SW).	
grades downward to 54.0	
Wet gray gravelly (SAND) with 15 to 20% gravel, very fine to very coarse size sand, compact, loose when disturbed, stratified, (SW).	
grades downward to 56.0	
See next sheet.	

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

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24G04 HOLE NO. Bore Hole 03-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION –

Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/09/04 COMPLETED 08/11/04

DEPTH IN FT — BLOWS ON SAMPLER

DEPTH	SN	0/6	6/12	12/18	18/24	N
	31	36				
	20		22			47
				25		
					17	
	32	40				
	20		45			117
				72		
					79	
	33	36				
65	8		80			
				100/4		
	34	69				
	10		100/5			
		Core	Run	#1		
70						
		Core	Run	#2		
75						
		Core	Run	#3		
80						

DESCRIPTION AND CLASSIFICATION

Extremely moist gray gravelly (SAND) with 15 to 20% gravel, occasional cobble and boulder, very fine to very coarse size sand, very dense, loose when disturbed, weakly stratified, (SW).

clear transition to 61.0

Extremely moist brown gravelly (SILTY-SAND) with 15 to 20% mostly subangular gravel, occasional cobble below 66.0 feet, little silt, dense becoming very dense below 62.0 feet, massive soil structure, (SM).

67.8

Gray dolomite bedrock, effervesses only when etched, moderately hard, easily etched with knife, moderately fractured horizontally with gypsum deposits, several low angle fractures, dense to very fine crystalline, highly broken and fractures zone 71.0 to 71.5 feet.

78.5

Coring completed at 78.5 feet.

WATER TABLE AND REMARKS

Run #	Depth (ft)	Length (ft)	Rec (ft)	Rec %	RQD %
1	68.5 to 71.0	2.5	2.1	84%	35%
2	71.0 to 76.0	5.5	5.4	98%	37%
3	76.5 to 78.5	2.0	1.9	95%	36%

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

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24G04 HOLE NO. Bore Hole 04-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION –
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/11/04 COMPLETED 08/11/04

DEPTH IN FT	SN	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WATER TABLE AND REMARKS
								Advanced bore hole with 4 1/4" I.D. hollow stem auger casing without split spoon sampling.	
5									
10	VS-1								VS-1 Vane shear testing conducted between 10.0 to 11.5 foot depths.
	VS-1								
	VS-1								
	ST-1								ST-1 Shelby tube soil sample taken between 12.0 to 14.0 foot depths REC = 24/24
	ST-1								
	ST-1								Recovered 24 inches.
	ST-1								
15									
20									

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron, Geologist. (cis) SHEET 1 OF 2

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24G04 HOLE NO. Bore Hole 04-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/11/04 COMPLETED 08/11/04

DEPTH IN FT	SN	BLOWS ON SAMPLER 0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WATER TABLE AND REMARKS
								Advanced bore hole with 4 1/4" I.D. hollow stem auger casing without split spoon sampling.	
	VS-2								VS-2 Vane shear testing conducted between 20.5 to 22.0 foot depths.
	VS-2								
	VS-2								
	ST-2								ST-2 Shelby tube soil sample taken between 22.5 to 24.5 foot depths REC 24/24
	ST-2								
	ST-2								
	ST-2								Recovered 24 inches.
25									
	VS-3								VS-3 Vane shear testing conducted between 29.0 to 30.5 foot depths.
30	VS-3								
	VS-3								
	ST-3								ST-3 Shelby tube soil sample taken between 31.0 to 33.0 foot depths REC 24/24
	ST-3								
	ST-3							33.0	
	ST-3							Boring completed at 33.0 feet.	Recovered 24 inches.
35									
40									

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cjs)

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24G04 HOLE NO. OW 01-04 SURF. ELEVATION 587.08

PROJECT Tonawanda Creek Rd. slope failure (E. of Transit) LOCATION _
Town of Clarence, Erie Co., NY

CLIENT McMahon & Mann Consulting Engineers, P.C. DATE STARTED 08/11/04 COMPLETED 08/11/04

DEPTH IN FT	BLOWS ON SAMPLER SN	0/6	6/12	12/18	18/24	N	LITH	DESCRIPTION AND CLASSIFICATION	WELL	WATER TABLE AND REMARKS
								Note: Advanced bore hole with 4 1/4" hollow stem augers without split spoon sampling to 12.0 feet.	CONCRETE; 2" SCHEDULE 40 PVC RISER	(1) Flush mount protective cover installed in concrete.
									BENTONITE SEAL	← 2.0
5									#00N SIZE MORIE SAND PACK	← 5.5
									.010 SLOT 2" PVC SCREEN	← 7.0
10								12.0		← 12.0
								Boring completed at 12.0 feet.		
15										
										No water at completion.
20										

N=NUMBER OF BLOWS TO DRIVE 2 " SPOON 12 " WITH 140 lb. WT. FALLING 30 " PER BLOW

LOGGED BY Brian Bartron,Geologist. (cjs) SHEET 1 OF 1

APPENDIX B

VANE SHEAR TEST RESULTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

McMahon & Mann
Consulting Engineers, P.C.

APPENDIX B

VANE SHEAR TEST RESULTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

MMCE used a field vane shear device to measure the shear strength of the soft clay soils encountered in the test borings. MMCE completed the vane sheat tests (VST's) in test borings B2-04 and B4-04. The test depths are shown on the boring logs in Appendix A. The tests were completed in general accordance with the procedures described in ASTM D 2573.

MMCE inserted the field vane into the test boring through the hollow stem augers. Centralizing devices were used to maintain the field vane rods in the center of the augers. The field vane was extended into the soil past the tip of the augers. MMCE then assembled the calibrated drive head on the end of the augers and began the test.

The test involves applying a torque to the drive head at rotation rate of 1 revolution per 5 seconds (0.1 degrees per second). MMCE recorded the resistance at specified intervals. Following the initial test, MMCE rotated the field vane 10 revolutions, waited 5 minutes then measured the remolded strength of the soil.

The results of the field vane shear tests are attached and are summarized below. The sensitivity, defined as the ratio of the peak to the remolded strength is also summarized on the table. The data indicate that the soft clay soils tested at this site generally have low to moderate sensitivity. This indicates that strength loss will occur when these soils are disturbed.

Boring Designation	Depth (ft)	Peak Shear Strength (psf)	Remolded Shear Strength (psf)	Sensitivity
B2-04	14.8	228.1	94.5	2.4
B2-04	20.5	467.0	48.9	9.6
B2-04	27.0	363.8	34.8	10.5
B4-04	11.5	608.2	185.7	3.3
B4-04	22.0	505.0	89.1	5.7
B4-04	30.5	293.2	98.8	3.0

SB2-04 - 14.8 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

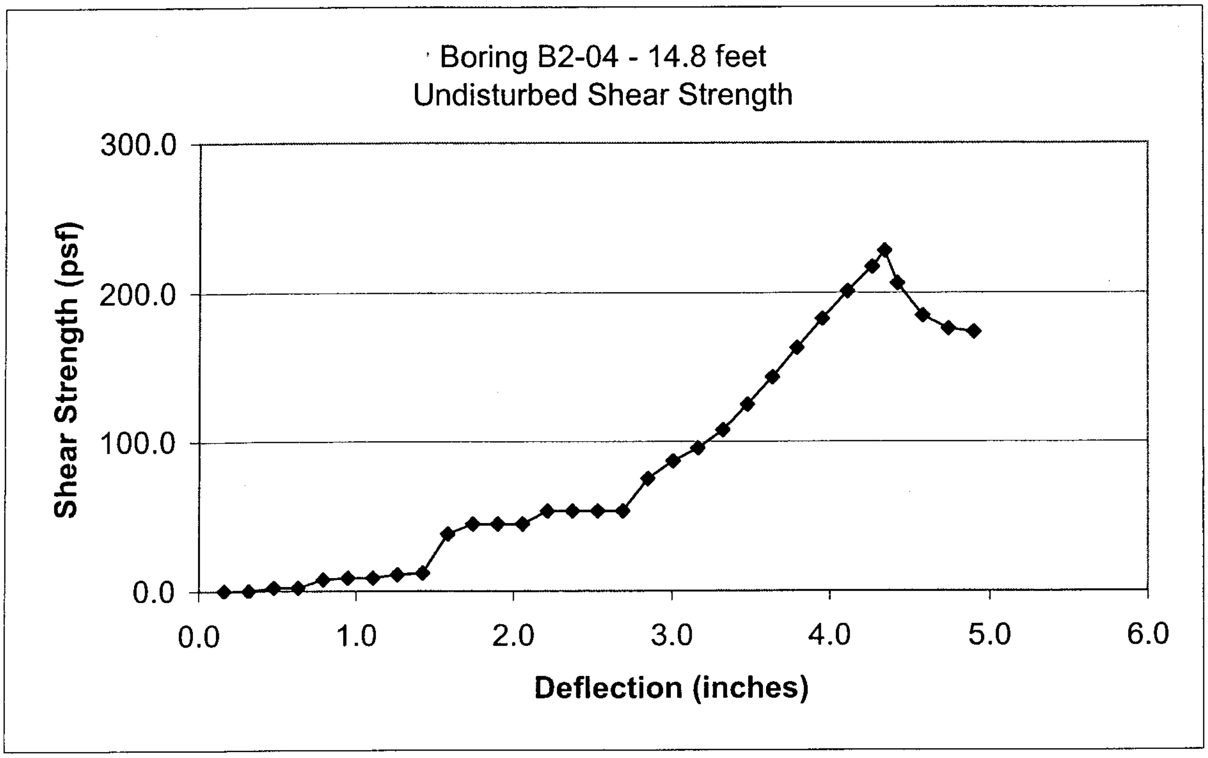
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	0.0	0.0
10	0.316	0.0	0.0
15	0.475	0.2	2.2
20	0.633	0.2	2.2
25	0.791	0.7	7.6
30	0.949	0.8	8.7
35	1.107	0.8	8.7
40	1.265	1.0	10.9
45	1.424	1.1	11.9
50	1.582	3.5	38.0
55	1.740	4.1	44.5
60	1.898	4.1	44.5
65	2.056	4.1	44.5
70	2.214	4.9	53.2
75	2.373	4.9	53.2
80	2.531	4.9	53.2
85	2.689	4.9	53.2
90	2.847	6.9	74.9
95	3.005	8.0	86.9
100	3.163	8.8	95.6
105	3.322	9.9	107.5
110	3.480	11.5	124.9
115	3.638	13.2	143.4
120	3.796	15.0	162.9
125	3.954	16.8	182.4
130	4.112	18.5	200.9
135	4.271	20.0	217.2
137.5	4.350	21.0	228.1
140	4.429	19.0	206.3
145	4.587	17.0	184.6
150	4.745	16.2	175.9
155	4.903	16.0	173.8

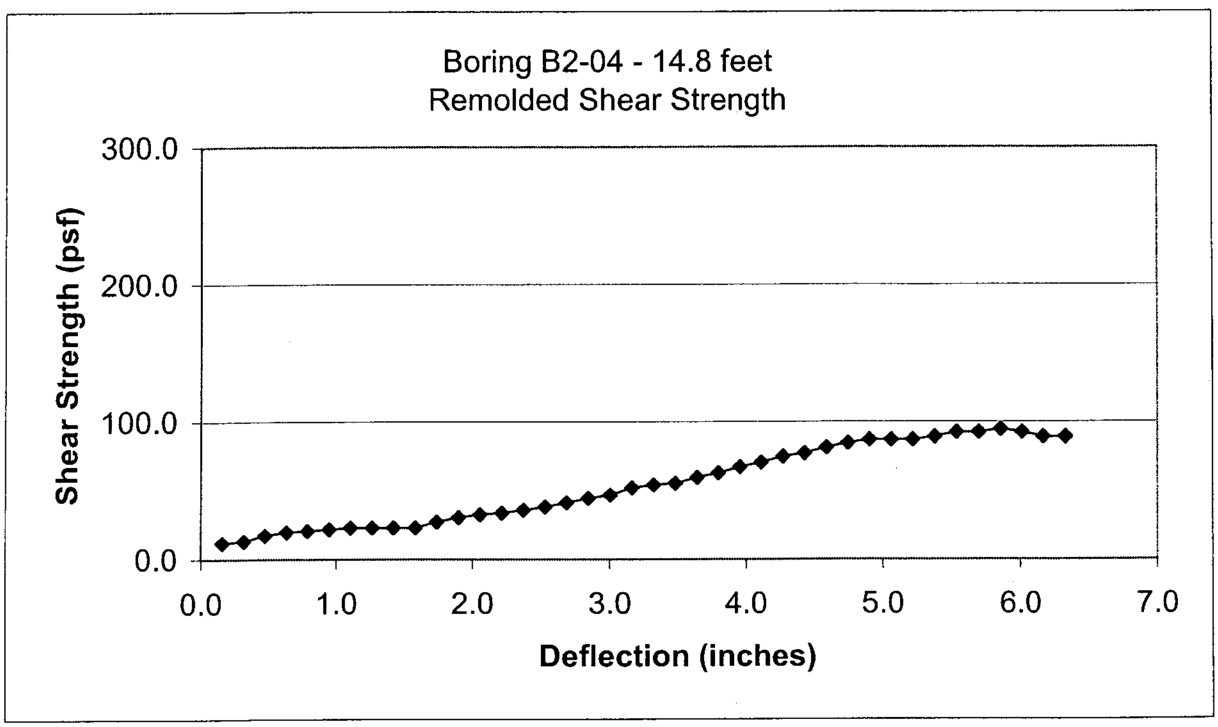
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	1.1	11.9
10	0.316	1.2	13.0
15	0.475	1.6	17.4
20	0.633	1.8	19.5
25	0.791	1.9	20.6
30	0.949	2	21.7
35	1.107	2.1	22.8
40	1.265	2.1	22.8
45	1.424	2.1	22.8
50	1.582	2.1	22.8
55	1.740	2.5	27.2
60	1.898	2.8	30.4
65	2.056	3	32.6
70	2.214	3.1	33.7
75	2.373	3.3	35.8
80	2.531	3.5	38.0
85	2.689	3.8	41.3
90	2.847	4.1	44.5
95	3.005	4.3	46.7
100	3.163	4.8	52.1
105	3.322	5	54.3
110	3.480	5.1	55.4
115	3.638	5.5	59.7
120	3.796	5.8	63.0
125	3.954	6.2	67.3
130	4.112	6.5	70.6
135	4.271	6.9	74.9
140	4.429	7.1	77.1
145	4.587	7.5	81.5
150	4.745	7.8	84.7
155	4.903	8	86.9
160	5.061	8	86.9
165	5.220	8	86.9
170	5.378	8.2	89.1
175	5.536	8.5	92.3
180	5.694	8.5	92.3
185	5.852	8.7	94.5
190	6.010	8.5	92.3
195	6.169	8.2	89.1
200	6.327	8.2	89.1



Ultimate Shear Strength (psf) = 228.1



Remolded Shear Strength (psf) = 94.5

SB2-04 - 20.5 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

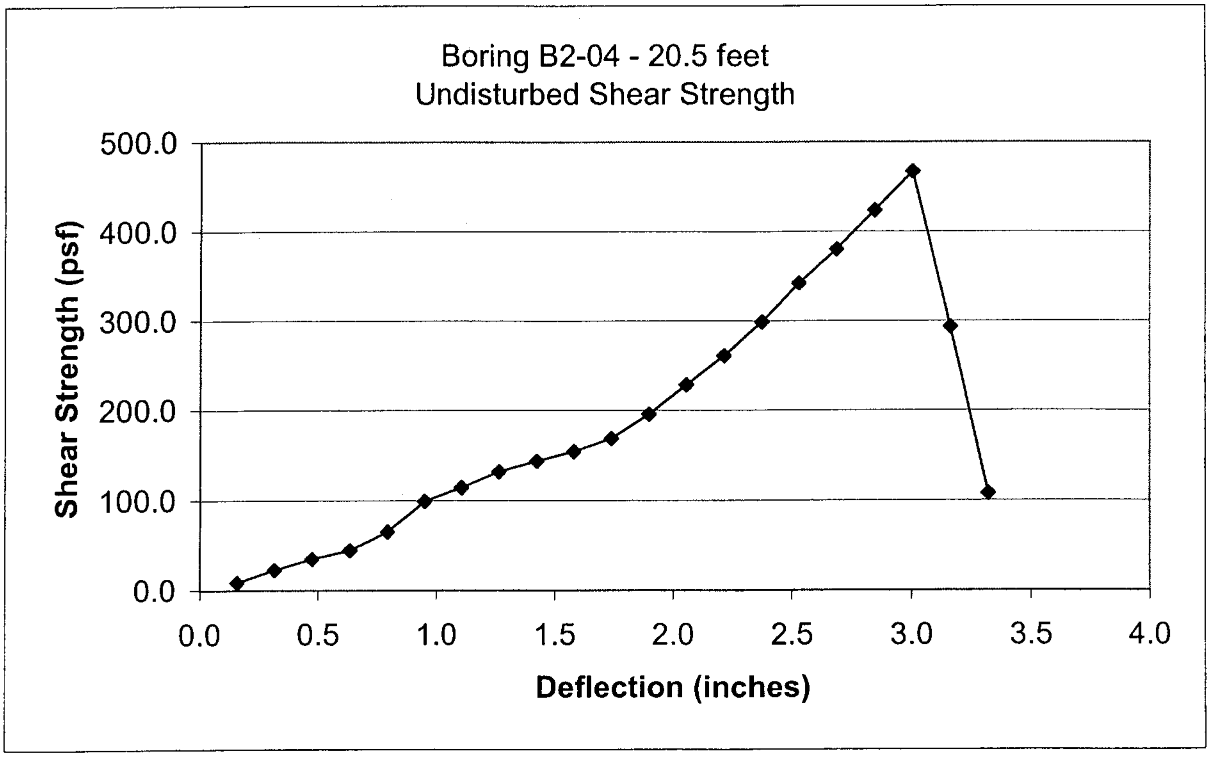
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	0.8	8.7
10	0.316	2.1	22.8
15	0.475	3.2	34.8
20	0.633	4.1	44.5
25	0.791	6.0	65.2
30	0.949	9.1	98.8
35	1.107	10.5	114.0
40	1.265	12.1	131.4
45	1.424	13.2	143.4
50	1.582	14.2	154.2
55	1.740	15.5	168.3
60	1.898	18.0	195.5
65	2.056	21.0	228.1
70	2.214	24.0	260.6
75	2.373	27.5	298.7
80	2.531	31.5	342.1
85	2.689	35.0	380.1
90	2.847	39.0	423.5
95	3.005	43.0	467.0
100	3.163	27.0	293.2
105	3.322	9.9	107.5

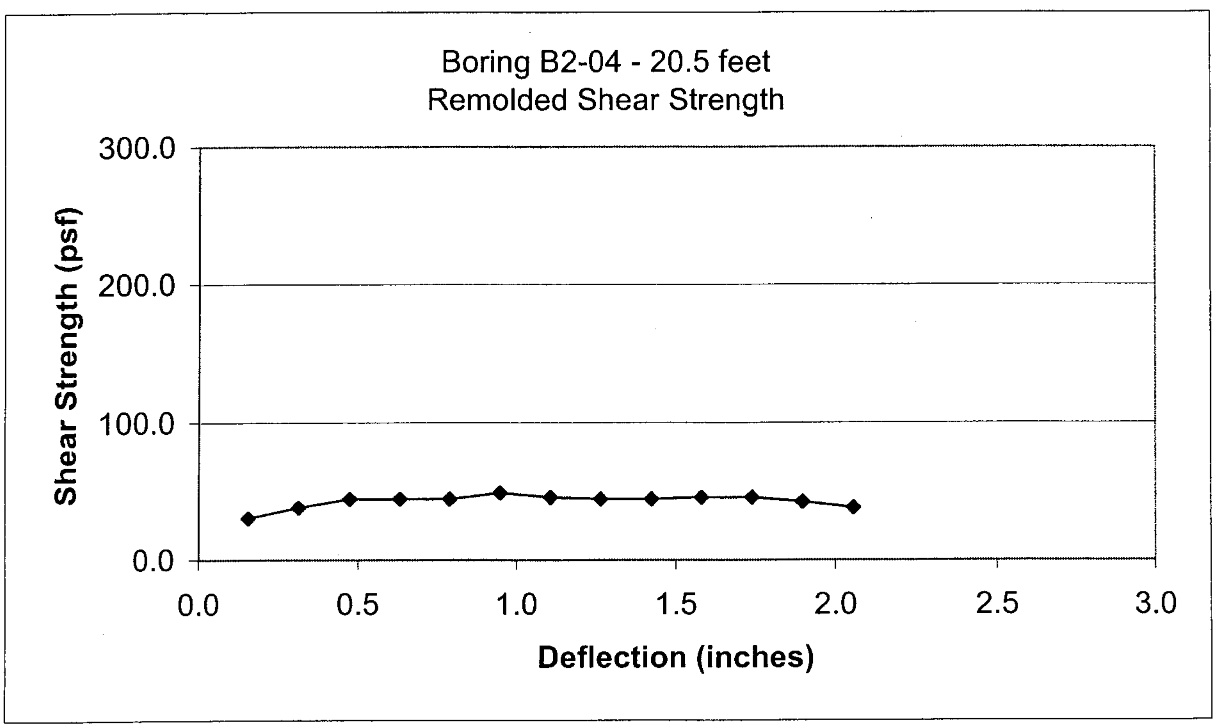
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	2.8	30.4
10	0.316	3.5	38.0
15	0.475	4.1	44.5
20	0.633	4.1	44.5
25	0.791	4.1	44.5
30	0.949	4.5	48.9
35	1.107	4.2	45.6
40	1.265	4.1	44.5
45	1.424	4.1	44.5
50	1.582	4.2	45.6
55	1.740	4.2	45.6
60	1.898	3.9	42.4
65	2.056	3.5	38.0



Ultimate Shear Strength (psf) = 467.0



Remolded Shear Strength (psf) = 48.9

SB2-04 - 27.0 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

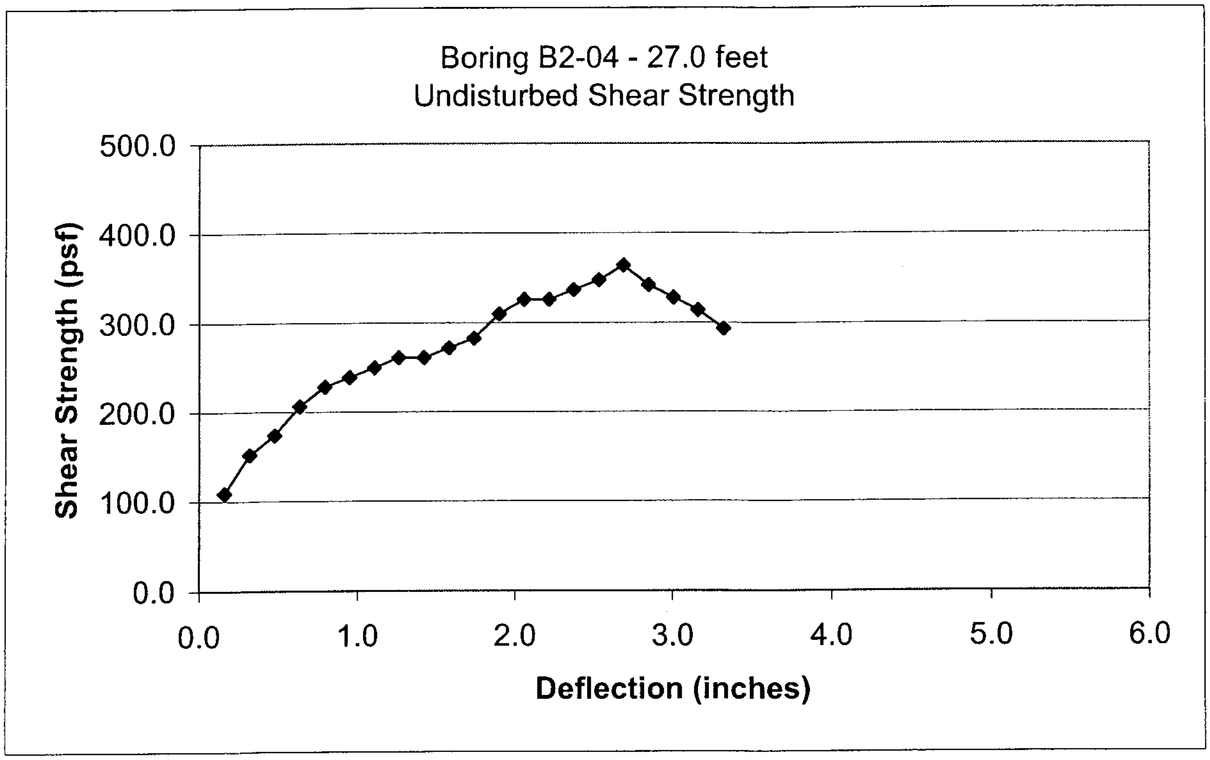
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	10.0	108.6
10	0.316	14.0	152.0
15	0.475	16.0	173.8
20	0.633	19.0	206.3
25	0.791	21.0	228.1
30	0.949	22.0	238.9
35	1.107	23.0	249.8
40	1.265	24.0	260.6
45	1.424	24.0	260.6
50	1.582	25.0	271.5
55	1.740	26.0	282.4
60	1.898	28.5	309.5
65	2.056	30.0	325.8
70	2.214	30.0	325.8
75	2.373	31.0	336.7
80	2.531	32.0	347.5
85	2.689	33.5	363.8
90	2.847	31.5	342.1
95	3.005	30.2	328.0
100	3.163	28.9	313.9
105	3.322	27.0	293.2

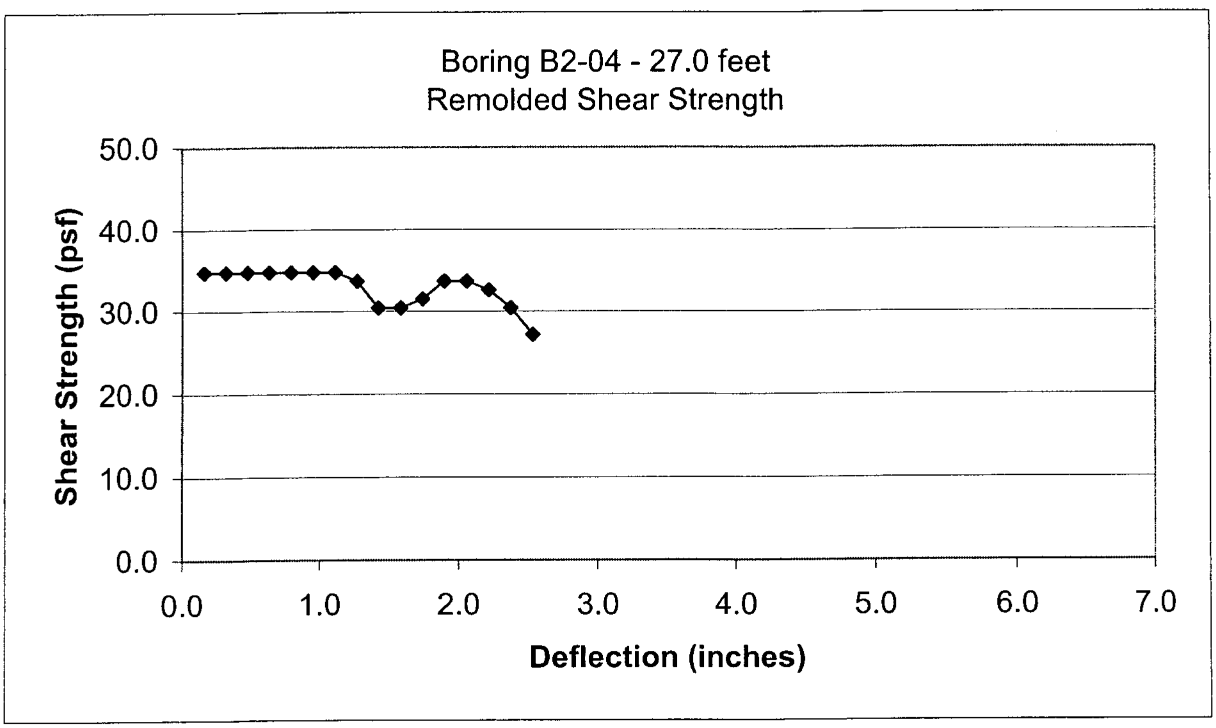
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	3.2	34.8
10	0.316	3.2	34.8
15	0.475	3.2	34.8
20	0.633	3.2	34.8
25	0.791	3.2	34.8
30	0.949	3.2	34.8
35	1.107	3.2	34.8
40	1.265	3.1	33.7
45	1.424	2.8	30.4
50	1.582	2.8	30.4
55	1.740	2.9	31.5
60	1.898	3.1	33.7
65	2.056	3.1	33.7
70	2.214	3	32.6
75	2.373	2.8	30.4
80	2.531	2.5	27.2



Ultimate Shear Strength (psf) = 363.8



Remolded Shear Strength (psf) = 34.8

SB4-04 - 11.5 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

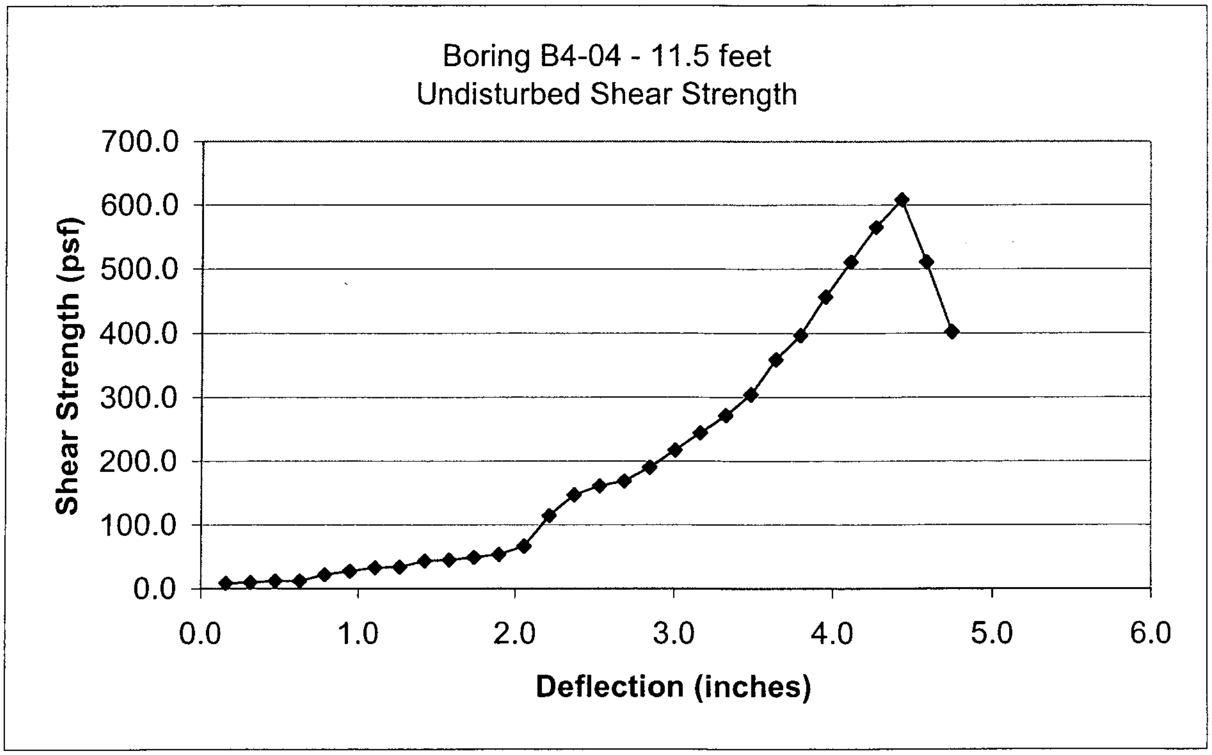
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	0.8	8.7
10	0.316	0.9	9.8
15	0.475	1.1	11.9
20	0.633	1.1	11.9
25	0.791	2.0	21.7
30	0.949	2.5	27.2
35	1.107	3.0	32.6
40	1.265	3.1	33.7
45	1.424	4.0	43.4
50	1.582	4.1	44.5
55	1.740	4.5	48.9
60	1.898	4.9	53.2
65	2.056	6.1	66.2
70	2.214	10.5	114.0
75	2.373	13.5	146.6
80	2.531	14.8	160.7
85	2.689	15.5	168.3
90	2.847	17.5	190.1
95	3.005	20.0	217.2
100	3.163	22.5	244.4
105	3.322	25.0	271.5
110	3.480	28.0	304.1
115	3.638	33.0	358.4
120	3.796	36.5	396.4
125	3.954	42.0	456.1
130	4.112	47.0	510.4
135	4.271	52.0	564.7
140	4.429	56.0	608.2
145	4.587	47.0	510.4
150	4.745	37.0	401.8

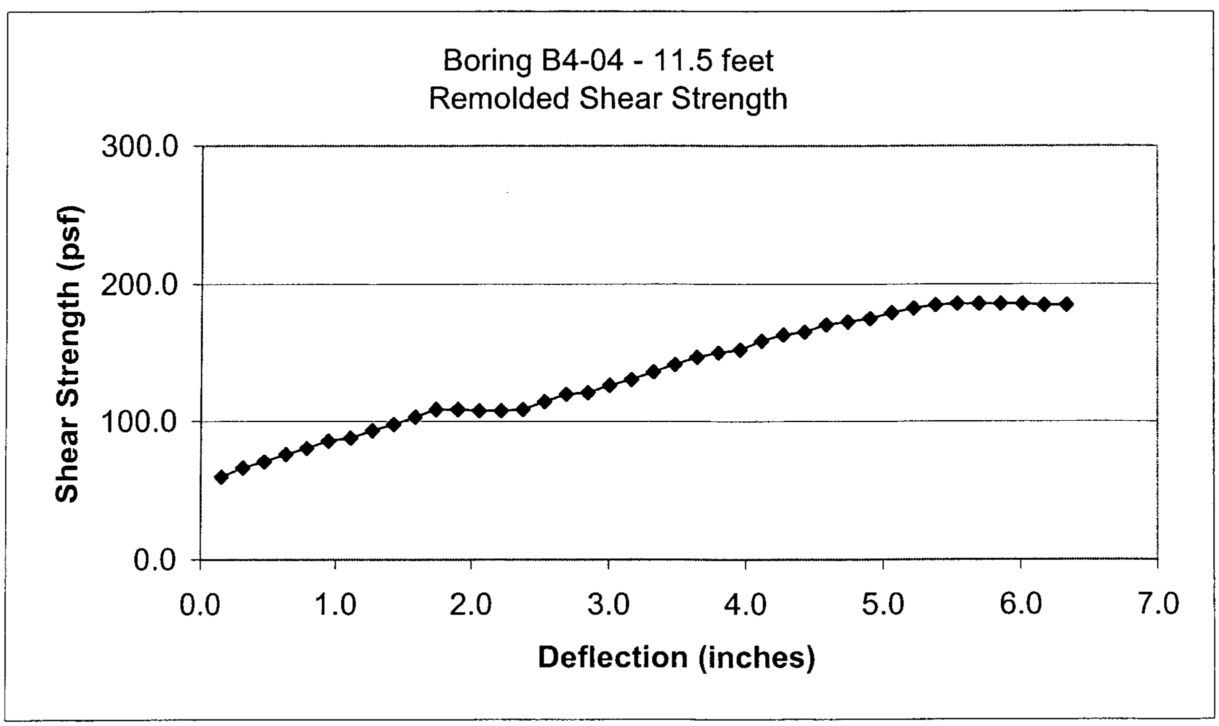
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	5.5	59.7
10	0.316	6.1	66.2
15	0.475	6.5	70.6
20	0.633	7	76.0
25	0.791	7.4	80.4
30	0.949	7.9	85.8
35	1.107	8.1	88.0
40	1.265	8.6	93.4
45	1.424	9	97.7
50	1.582	9.5	103.2
55	1.740	10	108.6
60	1.898	10	108.6
65	2.056	9.9	107.5
70	2.214	9.9	107.5
75	2.373	10	108.6
80	2.531	10.5	114.0
85	2.689	11	119.5
90	2.847	11.1	120.5
95	3.005	11.6	126.0
100	3.163	12	130.3
105	3.322	12.5	135.8
110	3.480	13	141.2
115	3.638	13.5	146.6
120	3.796	13.8	149.9
125	3.954	14	152.0
130	4.112	14.6	158.6
135	4.271	15	162.9
140	4.429	15.2	165.1
145	4.587	15.7	170.5
150	4.745	15.9	172.7
155	4.903	16.1	174.8
160	5.061	16.5	179.2
165	5.220	16.8	182.4
170	5.378	17	184.6
175	5.536	17.1	185.7
180	5.694	17.1	185.7
185	5.852	17.1	185.7
190	6.010	17.1	185.7
195	6.169	17	184.6
200	6.327	17	184.6



Ultimate Shear Strength (psf) = 608.2



Remolded Shear Strength (psf) = 185.7

SB4-04 - 22.0 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

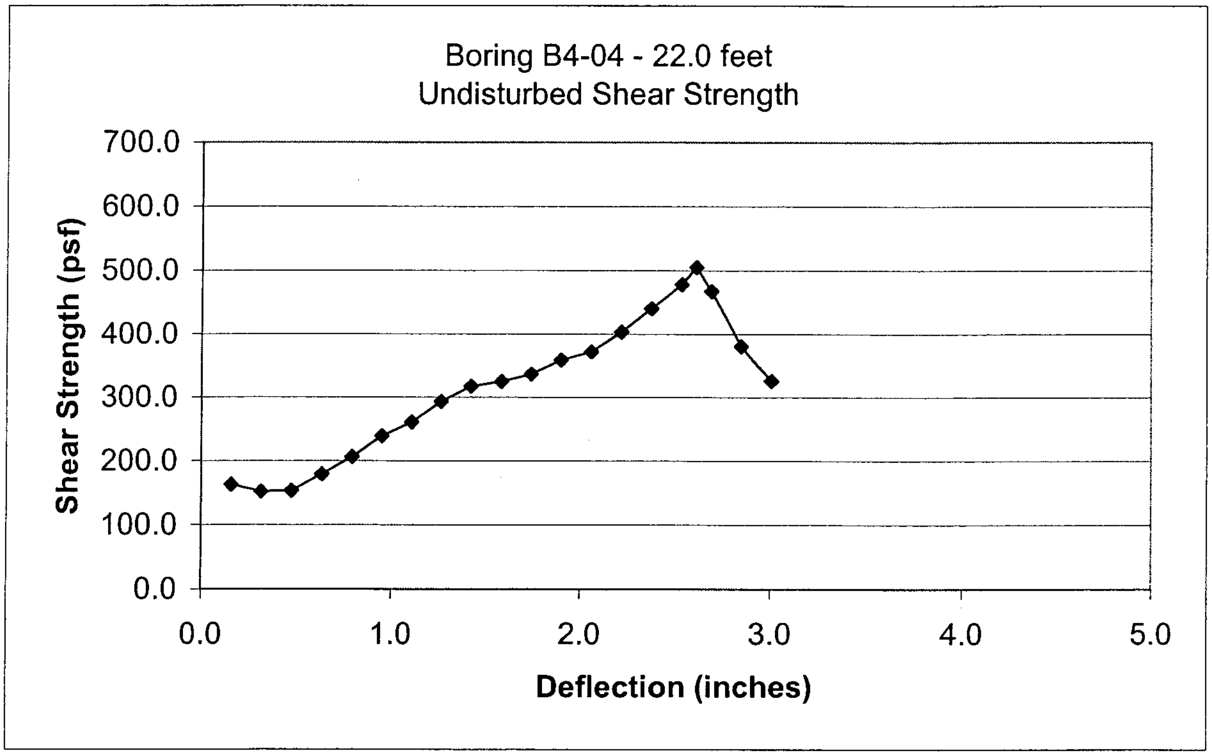
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	15.0	162.9
10	0.316	14.0	152.0
15	0.475	14.1	153.1
20	0.633	16.5	179.2
25	0.791	19.0	206.3
30	0.949	22.0	238.9
35	1.107	24.0	260.6
40	1.265	27.0	293.2
45	1.424	29.2	317.1
50	1.582	29.9	324.7
55	1.740	31.0	336.7
60	1.898	33.0	358.4
65	2.056	34.2	371.4
70	2.214	37.1	402.9
75	2.373	40.5	439.8
80	2.531	44.0	477.8
82.5	2.610	46.5	505.0
85	2.689	43.0	467.0
90	2.847	35.0	380.1
95	3.005	30.0	325.8

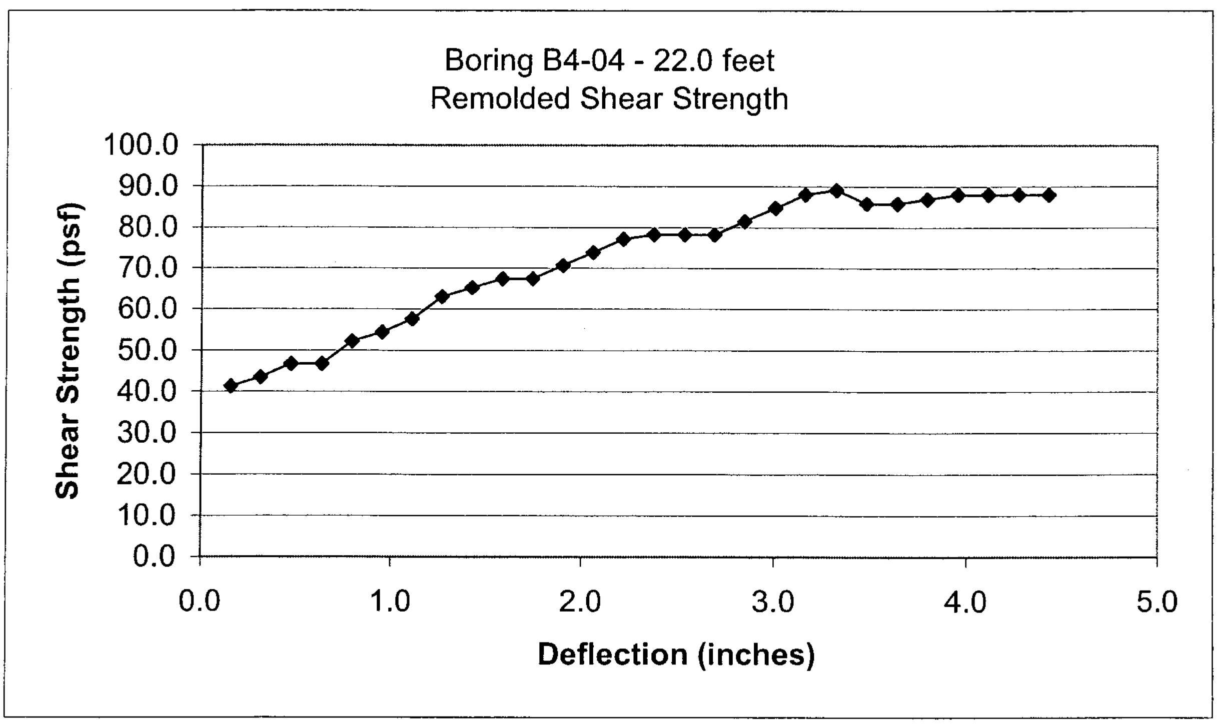
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	3.8	41.3
10	0.316	4	43.4
15	0.475	4.3	46.7
20	0.633	4.3	46.7
25	0.791	4.8	52.1
30	0.949	5	54.3
35	1.107	5.3	57.6
40	1.265	5.8	63.0
45	1.424	6	65.2
50	1.582	6.2	67.3
55	1.740	6.2	67.3
60	1.898	6.5	70.6
65	2.056	6.8	73.8
70	2.214	7.1	77.1
75	2.373	7.2	78.2
80	2.531	7.2	78.2
85	2.689	7.2	78.2
90	2.847	7.5	81.5
95	3.005	7.8	84.7
100	3.163	8.1	88.0
105	3.322	8.2	89.1
110	3.480	7.9	85.8
115	3.638	7.9	85.8
120	3.796	8	86.9
125	3.954	8.1	88.0
130	4.112	8.1	88.0
135	4.271	8.1	88.0
140	4.429	8.1	88.0



Ultimate Shear Strength (psf) = 505.0



Remolded Shear Strength (psf) = 89.1

SB4-04 - 30.5 feet deep

Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

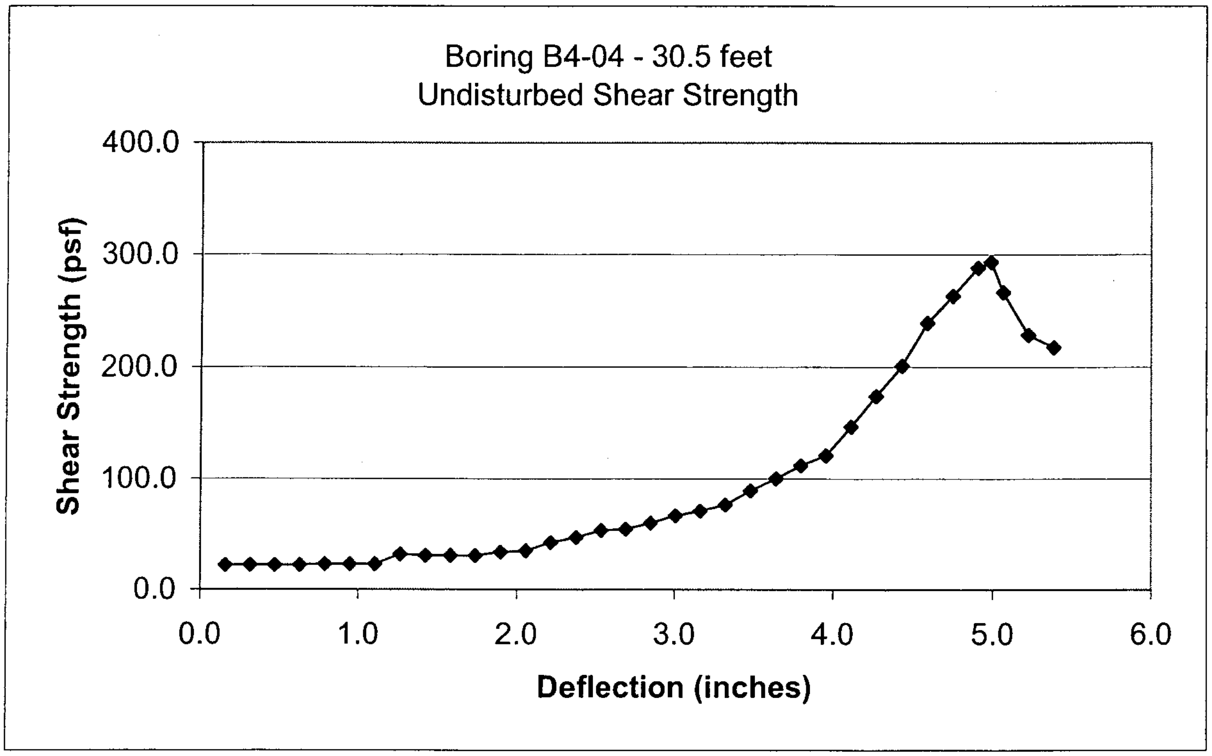
Undisturbed

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	2.0	21.7
10	0.316	2.0	21.7
15	0.475	2.0	21.7
20	0.633	2.0	21.7
25	0.791	2.1	22.8
30	0.949	2.1	22.8
35	1.107	2.1	22.8
40	1.265	2.9	31.5
45	1.424	2.8	30.4
50	1.582	2.8	30.4
55	1.740	2.8	30.4
60	1.898	3.1	33.7
65	2.056	3.2	34.8
70	2.214	3.9	42.4
75	2.373	4.3	46.7
80	2.531	4.9	53.2
85	2.689	5.0	54.3
90	2.847	5.5	59.7
95	3.005	6.1	66.2
100	3.163	6.5	70.6
105	3.322	7.0	76.0
110	3.480	8.2	89.1
115	3.638	9.2	99.9
120	3.796	10.3	111.9
125	3.954	11.1	120.5
130	4.112	13.5	146.6
135	4.271	16.0	173.8
140	4.429	18.5	200.9
145	4.587	22.0	238.9
150	4.745	24.2	262.8
155	4.903	26.5	287.8
157.5	4.982	27.0	293.2
160	5.061	24.5	266.1
165	5.220	21	228.1
170	5.378	20	217.2

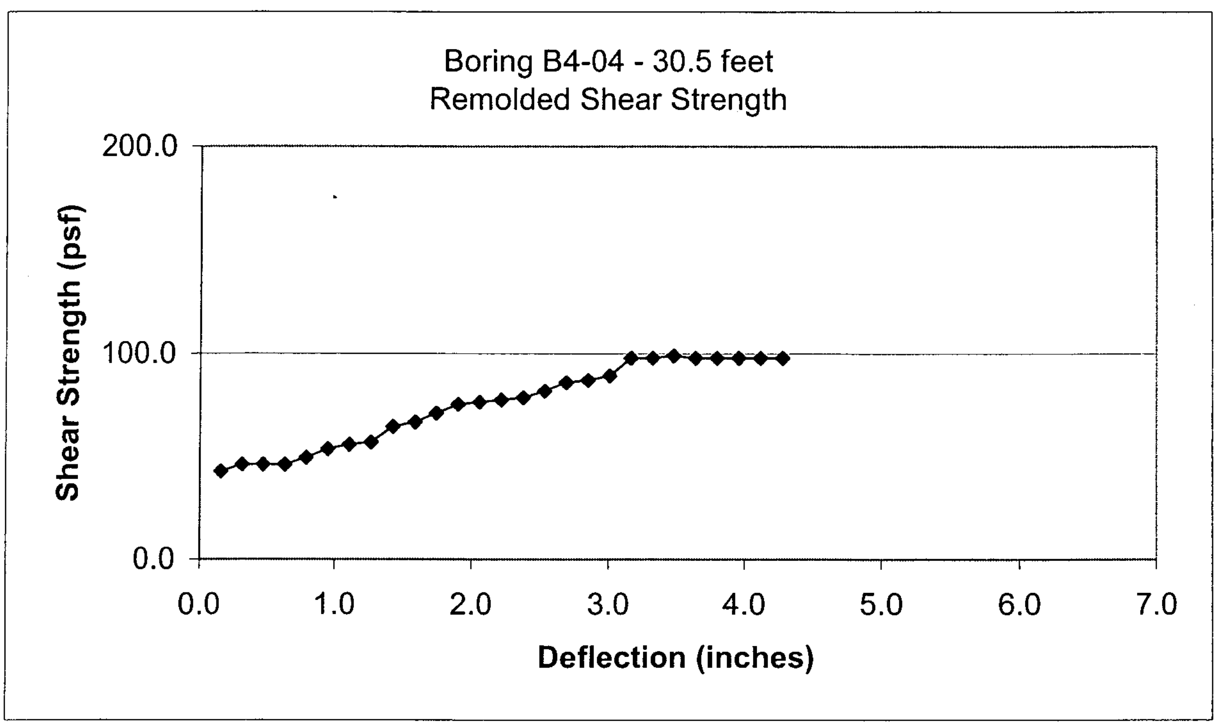
Torque Arm Length (inches) = 12
Vane Diameter (inches) = 3.625
Vane Constant = 0.905
Zero Reading (lbs) = 0

Remolded (1 minute)

Degrees of Rotation	Deflection (inches)	Dial Reading (lbs)	Shear Strength (psf)
5	0.158	3.9	42.4
10	0.316	4.2	45.6
15	0.475	4.2	45.6
20	0.633	4.2	45.6
25	0.791	4.5	48.9
30	0.949	4.9	53.2
35	1.107	5.1	55.4
40	1.265	5.2	56.5
45	1.424	5.9	64.1
50	1.582	6.1	66.2
55	1.740	6.5	70.6
60	1.898	6.9	74.9
65	2.056	7.0	76.0
70	2.214	7.1	77.1
75	2.373	7.2	78.2
80	2.531	7.5	81.5
85	2.689	7.9	85.8
90	2.847	8.0	86.9
95	3.005	8.2	89.1
100	3.163	9.0	97.7
105	3.322	9.0	97.7
110	3.480	9.1	98.8
115	3.638	9.0	97.7
120	3.796	9.0	97.7
125	3.954	9.0	97.7
130	4.112	9.0	97.7
135	4.271	9.0	97.7



Ultimate Shear Strength (psf) = 293.2



Remolded Shear Strength (psf) = 98.8

APPENDIX C

SUMMARY OF INCLINOMETER AND PIEZOMETER MEASUREMENTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

McMahon & Mann
Consulting Engineers, P.C.

APPENDIX C

SUMMARY OF INCLINOMETER AND PIEZOMETER MEASUREMENTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

I. INCLINOMETER

EDI installed an inclinometer in Boring B1-04 to monitor ground movements. The inclinometer consists of an approximately 2.75 inch diameter plastic pipe that extends from the ground surface to below the depth of potential ground movement. As shown on the log for boring B1-04, EDI set the tip of the inclinometer just below the top of rock. The casing is manufactured with two sets of grooves along the inside diameter of the pipe. The grooves are oriented 90 degrees apart. The inclinometer casing was installed such that one set of grooves (designated as the A-axis) is approximately perpendicular to the top edge of the failed slope and the other set (designated as the B-axis) is nearly parallel to the slope crest.

MMCE made measurements in the inclinometer by lowering a 2-foot long probe into the casing such that the probe is aligned along the grooves. Measurements of the probe inclination are made at 2-foot intervals. These measurements are summed to yield plots of horizontal deflection with depth.

The output data include plots of deflection along the A-axis and the B-axis. Reduced data also include plots of the vector sum of the A-axis and B-axis measurements with depth. This calculation allows the direction (azimuth) of the ground movement to be plotted with depth. The azimuth is the number of degrees measured clockwise from north.

A baseline reading was made in the inclinometer on August 6, 2004 and subsequent measurements were made August 9, 11, 13, 20 and 30, 2004. The results are summarized on the attached plot. The minor amount of movement indicated on the plot is believed to be associated with setting of the grout material around the plastic casing.

II. PIEZOMETER

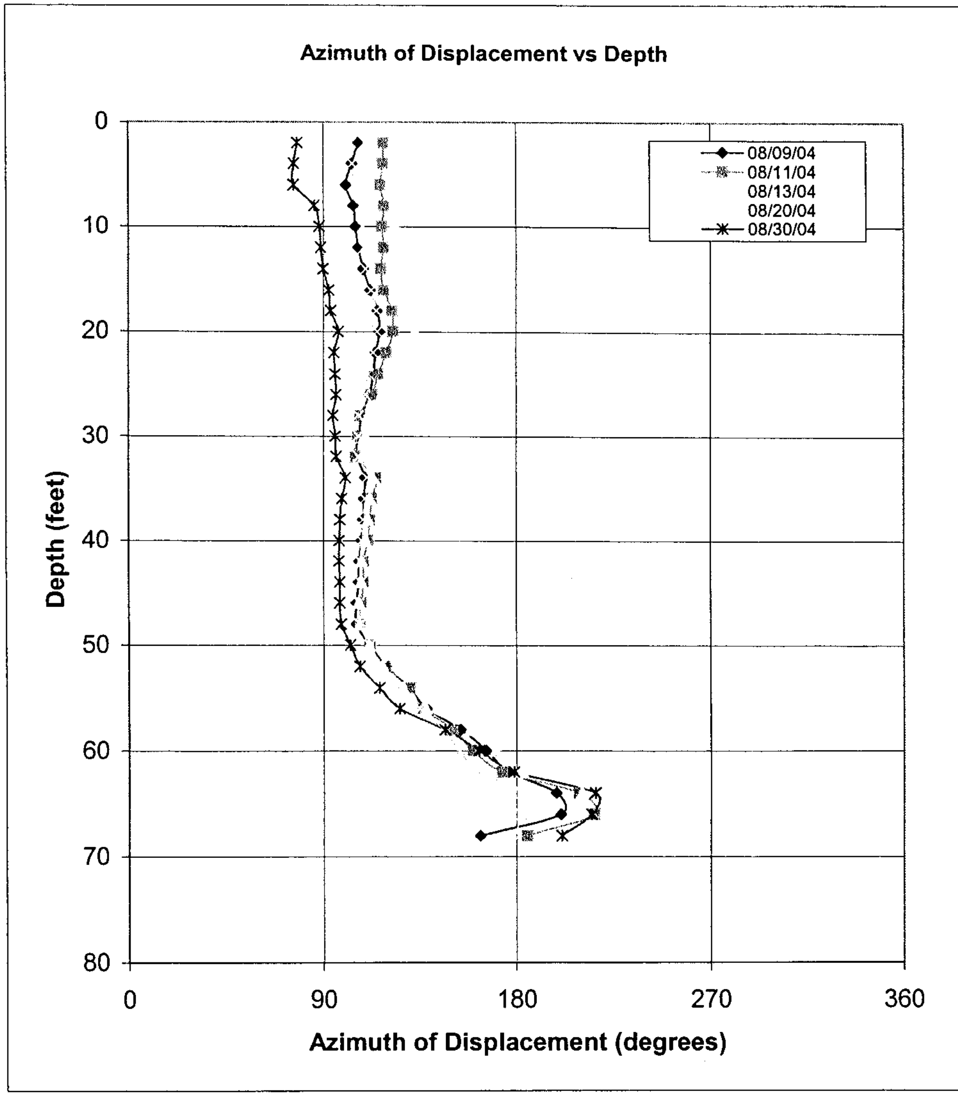
Groundwater level measurements were periodically recorded in piezometer OW 1-04 using an electronic tape water level reader. The readings were taken from the surveyed top of the road box. Groundwater elevations were calculated by subtracting the depth to water from the monitoring point elevation. The table below summarizes the data collected.

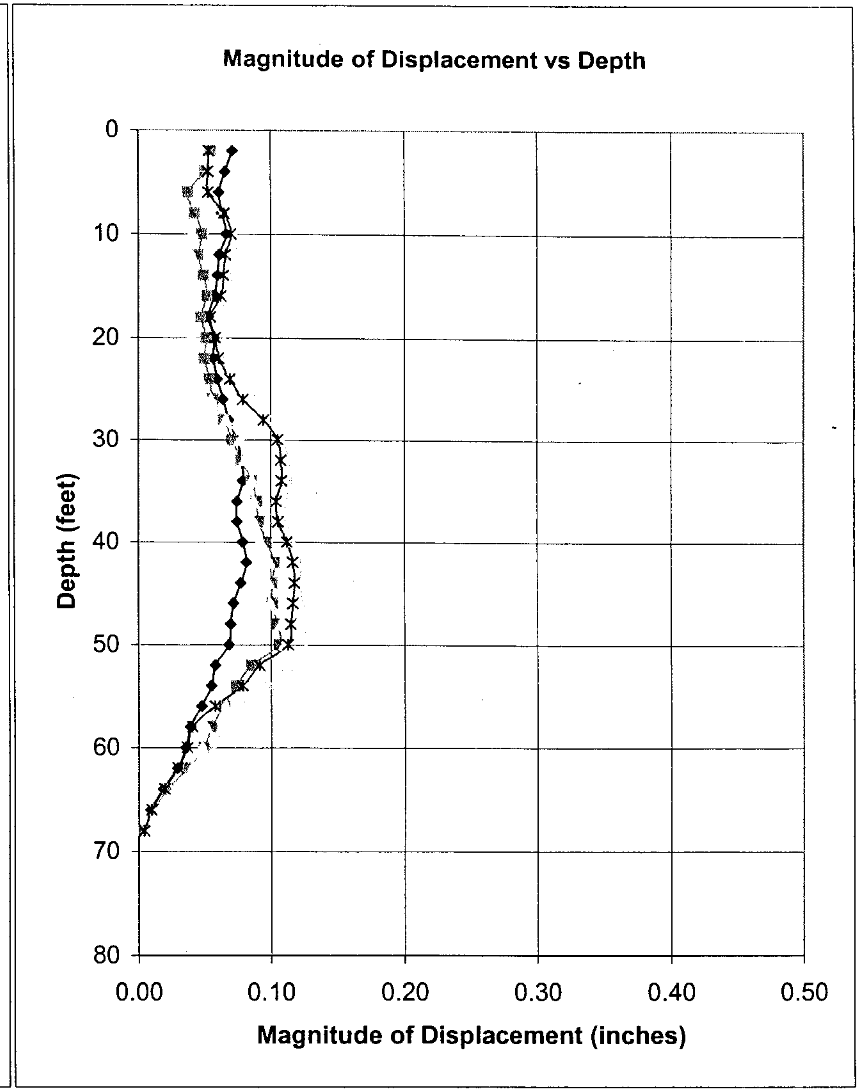
Date of Measurement	Water Elevation
August 13, 2004	Dry
August 20, 2004	Dry
October 8, 2004	579.4

Inclinometer I1-04 Data

Tonawanda Creek at Westphalinger Road

Baseline Date: 8/6/04





McMahon & Mann
Consulting Engineers, P.C.

APPENDIX D

SUMMARY OF LABORATORY TESTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

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APPENDIX D

SUMMARY OF LABORATORY TESTS
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

MMCE measured the moisture content of selected soil samples and engaged Geotechnics, of Pittsburgh, Pennsylvania, to measure the moisture content, Atterberg Limits, total unit weight and shear strength of soil samples. We also retained Geotesting Services, Inc., of Totowa, New Jersey, to complete classification tests on soft clay samples and to measure the strength of samples of the soft clay after mixing them with dry cement or a mixture of dry cement and lime. The laboratory testing is described in the following sections and the test data from Geotechnics and Geotesting are attached.

I. MOISTURE CONTENT

MMCE measured the moisture content of soil samples recovered from the test borings. The moisture content, defined as the ratio of moisture to dry soil weight, was measured in general accordance with ASTM method D2216. Geotechnics completed moisture content testing on one sample collected from Boring B3-04 and on Shelby Tube sample (Boring B-2, ST#2) and Geotesting measured the moisture content on Shelby tube samples from borings B2-04 and B4-04. The test results are summarized on the following table.

Boring No.	Sample Number	Sample Depth (ft)	Water Content (%)	Boring No.	Sample Number	Sample Depth (ft)	Water Content (%)
B1-04	S-2	2-4	19.8	B1-04	S-22	42-44	7.5
B1-04	S-3	4-6	23.2	B1-04	S-23	44-46	6.7
B1-04	S-4	6-8	22.2	B1-04	S-24	46-48	6.6
B1-04	S-5	8-10	20.8	B1-04	S-26	50-52	6.7
B1-04	S-6	10-12	37.7	B1-04	S-27	52-54	7.3
B1-04	S-7	12-14	41.6	B1-04	S-28	54-54.8	17.9
B1-04	S-8	14-16	47.7	B2-04	S-1	11-13	41.4
B1-04	S-9	16-18	48.9	B2-04	ST-1	15.25	38.1
B1-04	S-10	18-20	48.1	B2-04	ST-1	15.9	47.2
B1-04	S-11	20-22	43.5	B2-04	ST-1	16.5	49.8
B1-04	S-12	22-24	47.4	B2-04	ST-1	17	53.3
B1-04	S-13	24-26	47.5	B2-04	ST-2	22.4-22.8	52.2
B1-04	S-14	26-28	36.5	B2-04	ST-2	22.8-23.0	56.8
B1-04	S-15	28-30	40.6	B2-04	ST-3	28.2	50.0
B1-04	S-16	30-32	36.9	B2-04	ST-3	28.7	44.3
B1-04	S-17	32-34	14.0	B2-04	ST-3	29.2	43.0
B1-04	S-18	34-36	12.8	B2-04	ST-3	29.8	41.0
B1-04	S-19	36-38	9.7				
B1-04	S-20	38-40	9.7				
B1-04	S-21	40-42	14.7				

Boring No.	Sample Number	Sample Depth (ft)	Water Content (%)	Boring No.	Sample Number	Sample Depth (ft)	Water Content (%)
B3-04	S-1	1-2	3.7	B3-04	S-20	38-40	11.7
B3-04	S-2	2-4	17.9	B3-04	S-21	40-42	10.3
B3-04	S-3	4-6	22.4	B3-04	S-22	42-44	11.3
B3-04	S-4	6-8	24.3	B3-04	S-23	44-46	18.2
B3-04	S-5	8-10	19.9	B3-04	S-24	46-48	10.1
B3-04	S-6	10-12	35.2	B3-04	S-25	48-50	9.0
B3-04	S-7	12-14	45.2	B3-04	S-26	50-52	9.3
B3-04	S-8	14-16	56.2	B3-04	S-27	52-54	8.2
B3-04	S-9	16-18	48.4	B3-04	S-28	54-56	13.4
B3-04	S-10	18-20	52.4	B3-04	S-29	56-58	7.2
B3-04	S-11	20-22	57.6	B3-04	S-30	58-60	9.6
B3-04	S-12	22-24	46.7	B3-04	S-31	60-62	11.4
B3-04	S-13	24-25	39.6	B3-04	S-32	62-64	9.7
B3-04	S-14	26-28	44.0	B3-04	S-33	64-66	9.4
B3-04	S-15	28-30	40.9	B3-04	S-34	66-68	8.1
B3-04	S-16	30-32	40.6	B4-04	ST-2	23.4	52.2
B3-04	S-17	32-34	43.4	B4-04	ST-2	23.9	49.8
B3-04	S-18	34-36	12.2	B4-04	ST-2	24.2	56.6
B3-04	S-19	36-38	10.7	B4-04	ST-2	24.4	44.2

Note: ST denotes Shelby Tube Sample.

II. ATTERBERG LIMITS AND GRADATION TESTING

Geotechnics and Geotesting measured the Atterberg Limits on soil samples recovered from the test borings. The testing was completed in general accordance with ASTM method D 4318. The results are attached and are summarized below.

Boring No.	Sample Number	Sample Depth (ft)	Water Content (%)	LL (%)	PL (%)	PI (%)	LI
B1-04	S-7	12-14	41.6	35	18	17	1.4
B1-04	S-12	22-24	47.4	39	20	19	1.4
B1-04	S-17	32-34	14.0	Non-	Plastic		
B3-04	S-8	14-16	56.2	49	22	27	1.3
B3-04	S-15	28-30	40.9	40	20	20	1.1
B2-04	ST-2	22.8-23	56.9	45	20	25	1.5
Composite Sample	B2-04, ST-1, ST-3, B4-04, ST-2		44.2	46	19	27	.93

Note: (1) LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index and LI = Liquidity Index. Refer to Section V for discussion of Liquidity Index.

Additionally, Geotesting measured the gradation of the composite sample of the soft clay described in the previous table. The gradation plot is included with Geotesting's test data. The sample had 99.2 percent passing the No. 200 sieve and 56 percent finer than 2 microns.

III. UNIT WEIGHT

Geotechnics and Geotesting measured the total unit weight of Shelby tube samples by measuring the dimensions of a select portion of the Shelby tube and the weight of soil in that portion of the Shelby tube. The results are summarized below.

Boring No.	Sample No.	Sample Depth, Ft.	Total Unit Weight, pcf(1)	Moisture Content(2)	Dry Unit Weight pcf
B3-04	ST-2	22.4-22.8	105.8	52.2	69.5
B2-04	ST-1	15-17	109.3	47.1	74.3
B2-04	ST-3	28-30	112.9	44.6	78.1
B4-04	ST-2	22.5-24.5	109.0	50.7	72.3

Note (1) "pcf" means pounds per cubic foot.
(2) The moisture content is the average of the values measured for the Shelby tube sample.

IV. LABORATORY STRENGTH MEASUREMENTS

Direct Shear Test

Geotechnics measured the peak and residual shear strength of a Shelby tube sample collected from Boring B-3 at a depth of 22.4 to 22.8 feet. Geotechnics followed the Direct Shear Test Method, ASTM D 3080. The results are attached and indicate a peak angle of internal friction of 14.6 degrees and a residual angle of internal friction of 11.6 degrees.

Laboratory Vane Tests

Geotesting used a laboratory vane to measure the shear strength of Shelby tube samples. The test results are summarized in the following table.

Boring Designation	Depth (ft)	Peak Shear Strength (psf)	Remolded Shear Strength (psf)	Sensitivity
B2-04 ST-1	15.25	188	83	2.3
	15.9	121	59	2.1
	16.5	385	67	5.7
	17	315	62	5.1
B2-04 ST-3	28.2	159	48	3.3
	28.7	221	48	4.6
	29.2	194	65	3.0
	29.8	210	46	4.6
B4-04 ST-2	23.4	143	65	2.2
	23.9	320	48	6.7
	24.4	299	46	6.5
	24.2	358	54	6.6

Laboratory Tests on Mixed Samples

In addition to testing the soft clay, MMCE requested that Geotesting add dry cement and a combination of dry cement and lime to the soft clay, allow the mixture to cure and then measure the compressive strength of the mixture after 7, 14, 28 and 56 days. The testing is ongoing and the results received to date from Geotesting are attached.

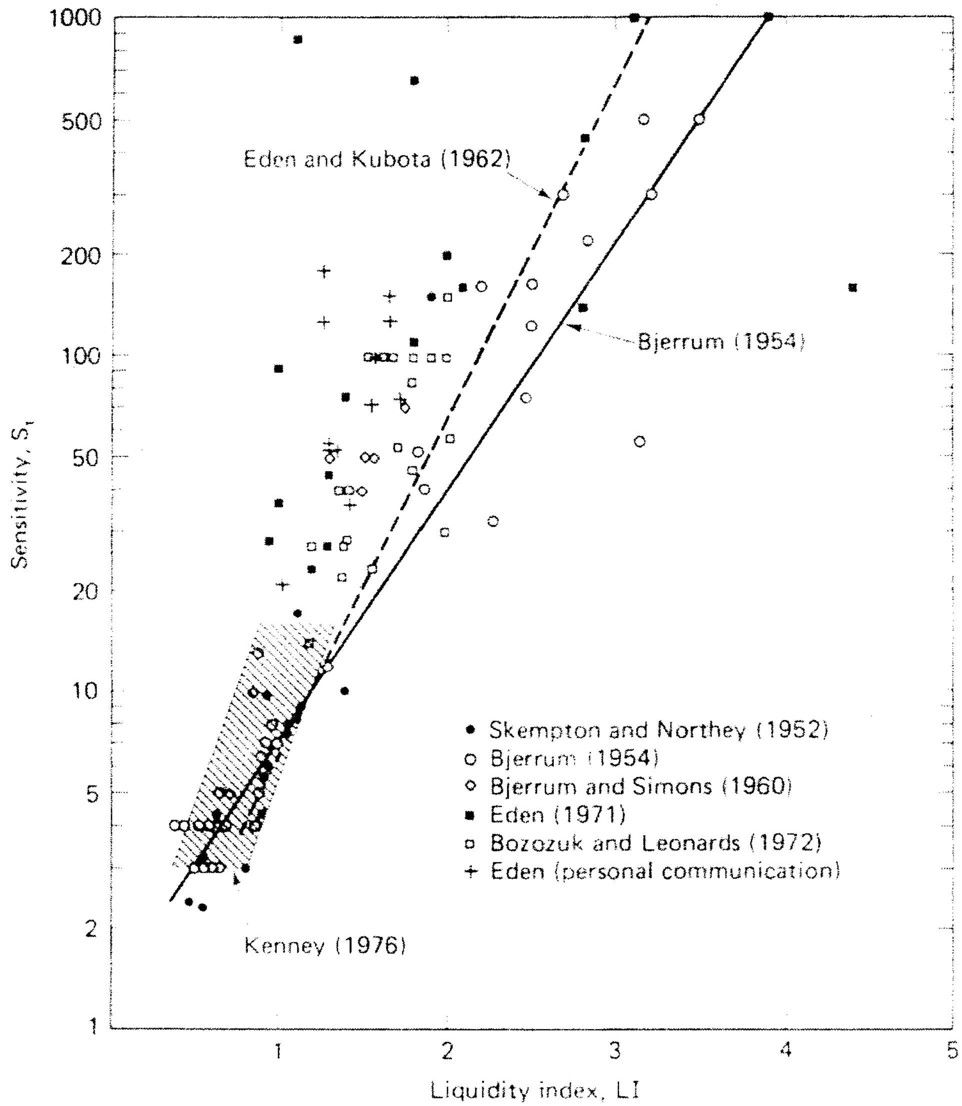
V. DISCUSSION

Review of the laboratory and field test data shows that it follows established correlations between sensitivity and soil characteristics. The soft clay samples exhibit a sensitivity ranging from about 2 to 7, as indicated on the table summarizing the laboratory vane test data. This is consistent with the test data from the field vane tests (see Appendix B) that indicate a sensitivity between about 2 and 10.

The plot on the following page from Holtz and Kovacs[1] shows a correlation between liquidity index (LI) and soil sensitivity. The liquidity index is defined as the ratio of the natural water content minus the plastic limit to the plasticity index and provides scale for comparison of a clay's natural water content to its Atterberg Limits.

[1] Holtz, R.D., and Kovacs, W.D., "An Introduction to Geotechnical Engineering," Prentice Hall, 1981.

The Atterberg Limit data summary table indicates that the liquidity index varies from about 0.9 to 1.5. As shown on the plot, this corresponds to a clay sensitivity that agrees well with values measured on clay samples from this site.



GEOTECHNICS TEST DATA

McMahon & Mann
Consulting Engineers, P.C.

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INTEGRITY IN TESTING

ATTERBERG LIMITS

ASTM D 4318-98 / AASHTO T89 (SOP - S4A)

Client	McMAHON & MANN	Boring No.	B-1
Client Reference	Towanda Creek @ W Phalinger	Depth (ft)	12-14
Project No.	2004-237-01	Sample No.	S-7
Lab ID	2004-237-01-01	Soil Description	**BROWN LEAN CLAY**

Note: The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description. (Minus No. 40 sieve material, Airdried)

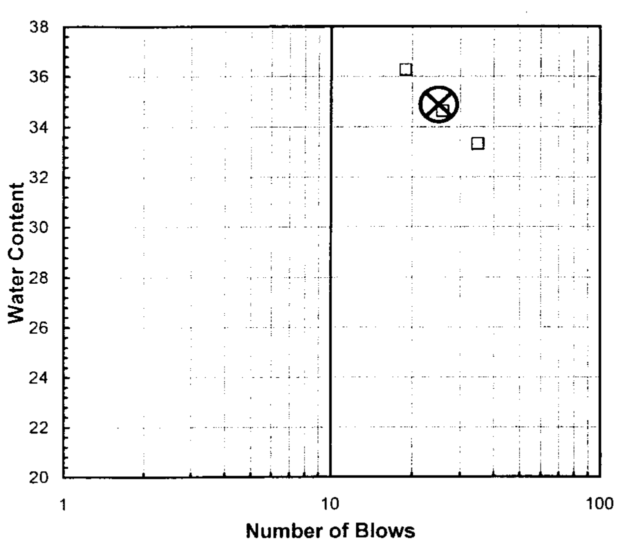
Liquid Limit Test	1	2	3	
				M
Tare Number	119	254	1882	U
Wt. of Tare & WS (gm)	38.42	42.47	41.89	L
Wt. of Tare & DS (gm)	33.24	~~36.28~~	35.86	T
Wt. of Tare (gm)	17.69	18.39	19.23	I
Wt. of Water (gm)	5.2	6.2	6.0	P
Wt. of DS (gm)	15.6	17.9	16.6	O
				I
Moisture Content (%)	**33.3**	**34.6**	**36.3**	N
Number of Blows	**35**	**26**	**19**	T

Plastic Limit Test	1	2	Range
Tare Number	2234	2298	
Wt. of Tare & WS (gm)	22.13	24.26	
Wt. of Tare & DS (gm)	21.20	23.30	
Wt. of Tare (gm)	16.11	17.92	
Wt. of Water (gm)	0.9	1.0	
Wt. of DS (gm)	5.1	5.4	
Moisture Content (%)	**18.3**	**17.8**	**0.4**

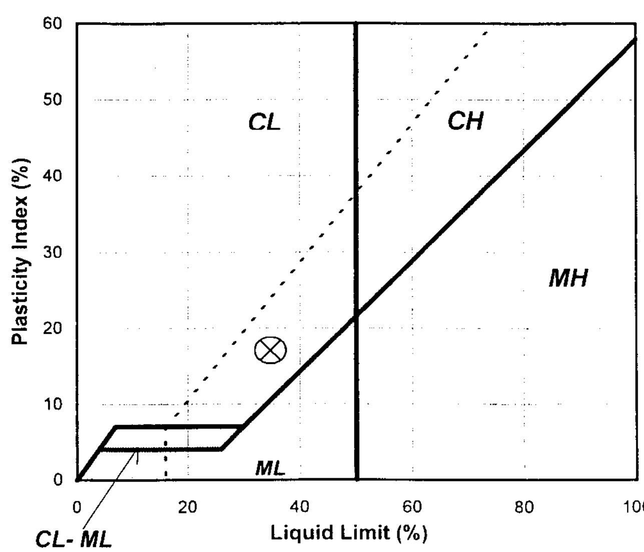
Note: The acceptable range of the two Moisture contents is ± 2.6

Test Results	
Liquid Limit (%)	**35**
Plastic Limit (%)	**18**
Plasticity Index (%)	**17**
USCS Symbol	**CL**

Flow Curve



Plasticity Chart



Tested By	BS	*Date*	08/09/04	*Checked By* KBL	*Date* 8/10/04

page 1 of 1 **DCN:** CT-S4B **DATE:** 10/08/01 **REVISION:** 2

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ATTERBERG LIMITS

ASTM D 4318-98 / AASHTO T89 (SOP - S4A)

Client	McMAHON & MANN	Boring No.	B-1
Client Reference	Towanda Creek @ W Phalinger	Depth (ft)	22-24
Project No.	2004-237-01	Sample No.	S-12
Lab ID	2004-237-01-02	Soil Description	**BROWN LEAN CLAY**

***Note: The* USCS *symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description*.** (Minus No. 40 sieve material, Airdried)

Liquid Limit Test	1	2	3
Tare Number	120	128	162
Wt. of Tare & WS (gm)	35.99	38.87	35.74
Wt. of Tare & DS (gm)	31.20	33.47	30.60
Wt. of Tare (gm)	18.45	19.77	18.14
Wt. of Water (gm)	4.8	5.4	5.1
Wt. of DS (gm)	12.8	13.7	12.5
Moisture Content (%)	**37.6**	**39.4**	**41.3**
Number of Blows	**32**	**25**	**17**

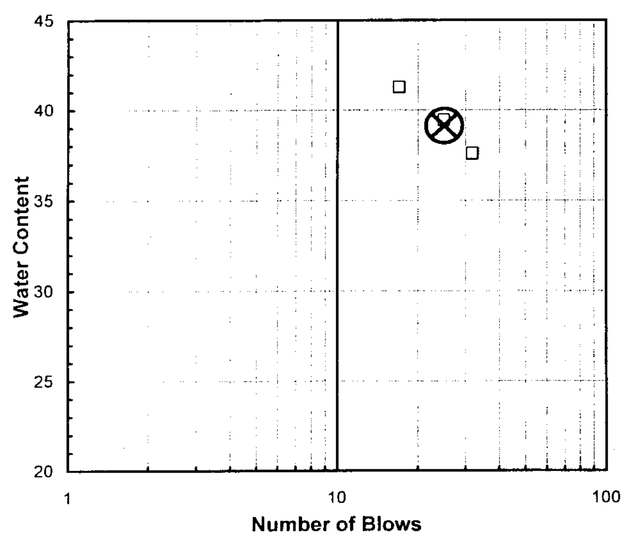
MULTIPOINT

Plastic Limit Test	1	2	Range
Tare Number	174	2254	
Wt. of Tare & WS (gm)	21.62	22.02	
Wt. of Tare & DS (gm)	20.64	21.16	
Wt. of Tare (gm)	15.64	16.83	
Wt. of Water (gm)	1.0	0.9	
Wt. of DS (gm)	5.0	4.3	
Moisture Content (%)	**19.6**	**19.9**	**-0.3**

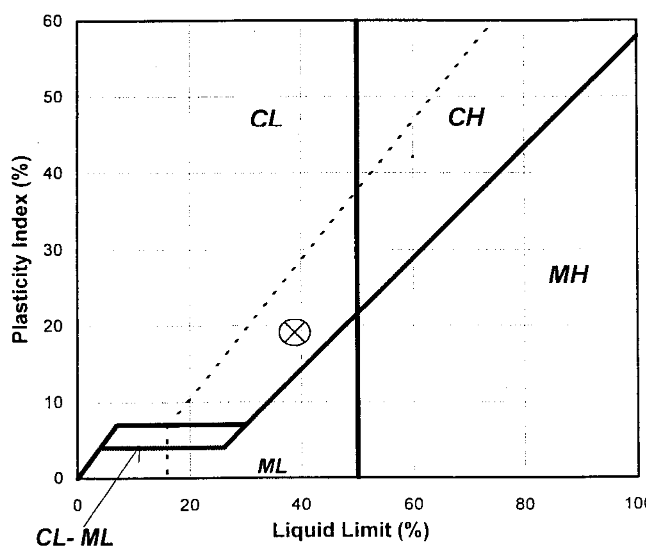
Note: The acceptable range of the two Moisture contents is ± 2.6

Test Results	
Liquid Limit (%)	**39**
Plastic Limit (%)	**20**
Plasticity Index (%)	**19**
USCS Symbol	**CL**

Flow Curve



Plasticity Chart



Tested By BS *Date* 08/09/04 *Checked By* [illegible] *Date* 8/10/04

 DCN: CT-S4B **DATE:** 10/08/01 **REVISION:** 2

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ATTERBERG LIMIT

ASTM D 4318-00/AASHTO T89-96, T90-00 (SOP - S4)

Client	McMAHON & MANN	Boring No.	B-1
Client Reference	Towanda Crk @ W. Phallinger 04-013	Depth (ft)	32-34
Project No.	2004-237-01	Sample No.	S-17
Lab ID	2004-237-01-03	Visual Description	**BROWN SILT**
			(MInus No. 40 sieve material, Airdried)

NON - PLASTIC MATERIAL

Tested By BS *Date* 08/09/04 *Checked By* Kel *Date* 8/9/04

 DCN: CT-S4C DATE: 7-11-97 REVISION : 2 *C:\MSOFFICE\Excel\Printq\[X933.XLS]Sheet1*

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MOISTURE CONTENT

ASTM D 2216 (SOP-S1)

Client	McMAHON & MANN
Client Reference	TOWANDA CREEK @ W. PHALINGER 04-013
Project No.	2004-237-02

Lab ID	02	03	03
Boring No.	B-3	B-2	B-2
Depth (ft)	28-30	22.8-23.0	22.4-22.8
Sample No.	S-15	ST-2	ST-2
Tare Number	575	444	1644
Wt. of Tare & WS (gm)	315.68	180.33	387.48
Wt. of Tare & DS (gm)	309.54	150.99	286.68
Wt. of Tare (gm)	294.52	99.44	93.55
Wt. of Water (gm)	6.14	29.34	100.8
Wt. of DS (gm)	15.02	51.55	193.13
Water Content (%)	**40.9**	**56.9**	**52.2**

Notes : *NA*

Tested By *DB* *Date* *08/30/04* *Checked By* Jm *Date* 12-2-04

DCN: CT-S1 DATE 6-30-98 REVISION: 2

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ATTERBERG LIMITS

ASTM D 4318-98 / AASHTO T89 (SOP - S4A)

Client	McMAHON & MANN	Boring No.	B-3
Client Reference	Towanda Creek @ W Phalinger 04-013	Depth (ft)	14-16
Project No.	2004-237-02	Sample No.	S-8
Lab ID	2004-237-02-01	Soil Description	GRAY LEAN CLAY

(Minus No. 40 sieve material, Airdried)

***Note:** The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description.*

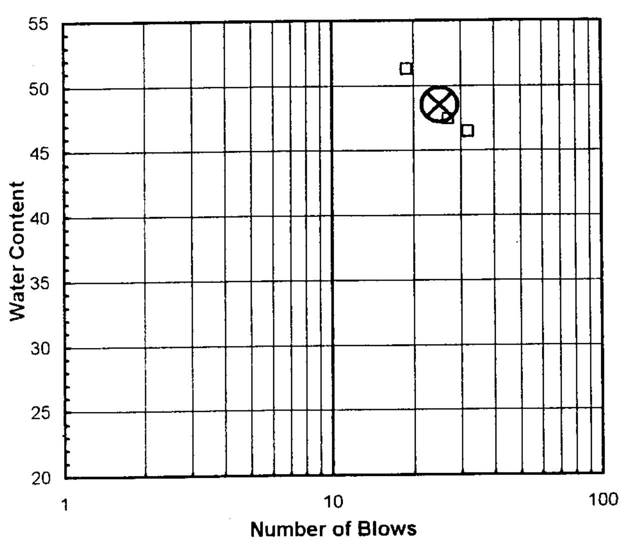
Liquid Limit Test	1	2	3	MULTIPOINT
Tare Number	2045	2227	2228	
Wt. of Tare & WS (gm)	37.85	39.69	34.33	
Wt. of Tare & DS (gm)	31.88	32.99	28.48	
Wt. of Tare (gm)	19.03	18.86	17.07	
Wt. of Water (gm)	6.0	6.7	5.9	
Wt. of DS (gm)	12.9	14.1	11.4	
Moisture Content (%)	**46.5**	**47.4**	**51.3**	
Number of Blows	**32**	**27**	**19**	

Plastic Limit Test	1	2	Range
Tare Number	2298	2301	
Wt. of Tare & WS (gm)	23.99	26.01	
Wt. of Tare & DS (gm)	22.89	24.88	
Wt. of Tare (gm)	17.94	19.80	
Wt. of Water (gm)	1.1	1.1	
Wt. of DS (gm)	5.0	5.1	
Moisture Content (%)	**22.2**	**22.2**	**0.0**

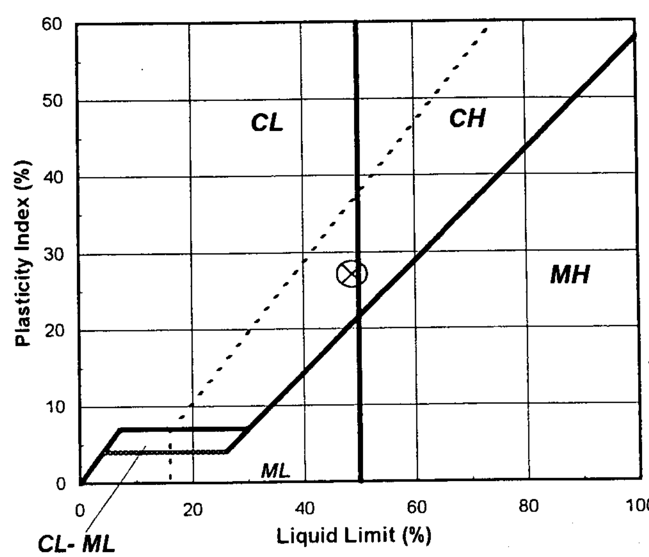
Note: The acceptable range of the two Moisture contents is ± 2.6

Test Results	
Liquid Limit (%)	**49**
Plastic Limit (%)	**22**
Plasticity Index (%)	**27**
USCS Symbol	**CL**

Flow Curve



Plasticity Chart



Tested By BS *Date* 08/30/04 *Checked By* T.O *Date* 9-1-04

 DCN: CT-S4B **DATE:** 10/08/01 **REVISION:** 2

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ATTERBERG LIMITS

ASTM D 4318-98 / AASHTO T89 (SOP - S4A)

Client	McMAHON & MANN	Boring No.	B-3
Client Reference	Towanda Creek @ W Phalinger 04-013	Depth (ft)	28-30
Project No.	2004-237-02	Sample No.	S-15
Lab ID	2004-237-02-02	Soil Description	**GRAY LEAN CLAY**

Note: The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description. (Minus No. 40 sieve material, Airdried)

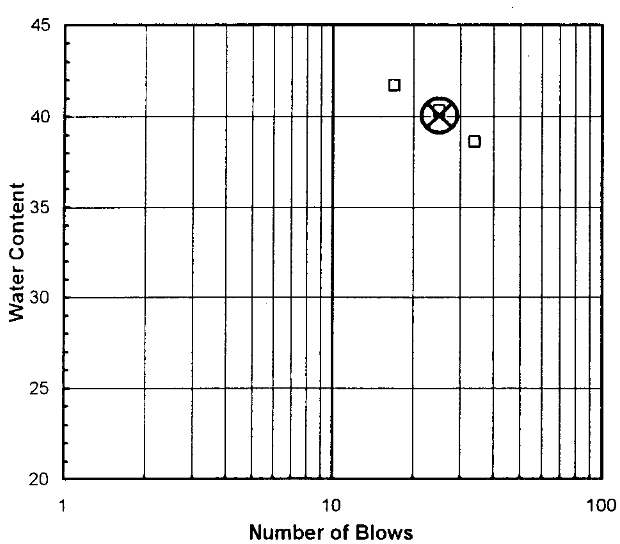
Liquid Limit Test	1	2	3	MULTIPOINT
Tare Number	119	162	174	
Wt. of Tare & WS (gm)	39.11	42.51	38.12	
Wt. of Tare & DS (gm)	33.15	35.51	31.51	
Wt. of Tare (gm)	17.71	18.15	15.66	
Wt. of Water (gm)	6.0	7.0	6.6	
Wt. of DS (gm)	15.4	17.4	15.9	
Moisture Content (%)	**38.6**	**40.3**	**41.7**	
Number of Blows	**34**	**25**	**17**	

Plastic Limit Test	1	2	Range
Tare Number	253	1168	
Wt. of Tare & WS (gm)	24.35	26.85	
Wt. of Tare & DS (gm)	23.28	25.51	
Wt. of Tare (gm)	17.87	18.58	
Wt. of Water (gm)	1.1	1.3	
Wt. of DS (gm)	5.4	6.9	
Moisture Content (%)	**19.8**	**19.3**	**0.4**

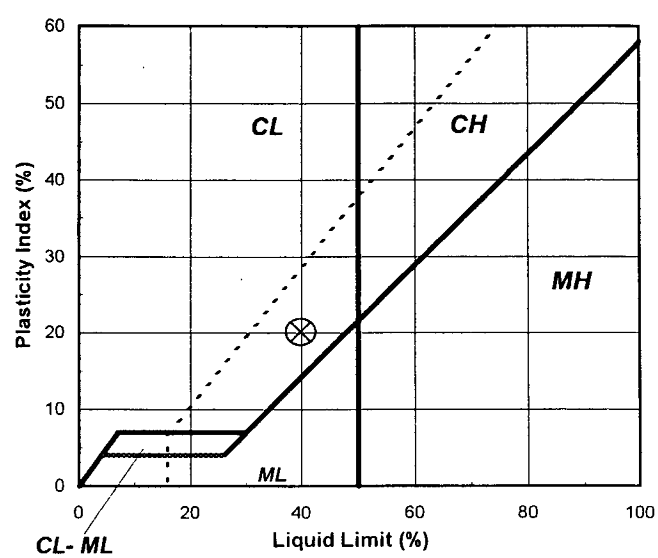
Note: The acceptable range of the two Moisture contents is ± 2.6

Test Results	
Liquid Limit (%)	**40**
Plastic Limit (%)	**20**
Plasticity Index (%)	**20**
USCS Symbol	**CL**

Flow Curve



Plasticity Chart



Tested By BS *Date* 08/30/04 *Checked By* TD *Date* 9-1-04

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SHELBY TUBE UNIT WEIGHT

(SOP - S37)

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	Towanda Creek @ W Phalinger 04-013	Depth Pushed(ft)	21.0-23.0
Project No.	2004-237-02	Shelby Tube No.	ST-2
Lab ID	2004-237-02-03	Recovery	14/24

MOISTURE CONTENT

Section Number	1	2	3	4	5
Tare Number	444	1644			
Wt. Tare & WS(gm.)	180.33	387.48			
Wt. Tare & DS(gm.)	150.99	286.68			
Wt. Tare(gm.)	99.44	93.55			
Moisture Content(%)	56.92	52.19			

UNIT WEIGHT

Wt. Tube & WS.(gms.)	1225.80
Wt. Of Tube(gms.)	350.60
Wt. Of WS.(gms.)	875.20
Length 1 (in.)	4.825
Length 2 (in.)	4.814
Length 3 (in.)	4.798
Top Diameter (in.)	2.885
Middle Diameter (in.)	2.888
Bottom Diameter (in.)	2.887
Sample Volume (cc)	516.01
Moisture Content(%)	52.19
Unit Wet Wt.(gms/cc)	1.70
Unit Wet Wt.(pcf.)	105.8
Unit Dry Wt.(gms/cc)	**1.11**
Unit Dry Wt.(pcf.)	**69.5**

SOIL PROFILE AND SAMPLING

DEPTH ( )	ELEV ( )	SECTION No.	SOIL PROFILE	SOIL DESCRIPTION AND REMARKS	TEST PERFORMED
21.0					
21.5					
22.0					
22.5		2		BROWN TO GRAYISH BROWN LEAN CLAY	UNIT WGT. WC DIRECT SHEAR PEAK & RESIDUAL
		1		GRAYISH BROWN AT THE BOTTOM	WC LIMITS
23.0					

Note: *When full recovery is not achieved, the elevation can not be accurately defined.*
Indicate each cut of the tube with an arrow.
Indicate dividing line between soil types with a solid line.
Indicate wax by cross-hatching. Indicate soil types by standard symbols.

Tested By *TM* Date *08/27/04* Checked By *Jem* Date *12-2-04*

 DCN: CT-S37 DATE:1-29-98 REVISION: 2 C:\MSOFFICE\Excel\Printing\A260.xls]Sheet1

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Geotechnics
INTEGRITY IN TESTING

ATTERBERG LIMITS

ASTM D 4318-98 / AASHTO T89 (SOP - S4A)

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	Towanda Creek @ W Phalinger 04-013	Depth (ft)	22.8-23.0
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Soil Description	GRAYISH BROWN LEAN CLAY
			(Minus No. 40 sieve material. Airdried)

Note: The USCS symbol used with this test refers only to the minus No. 40 sieve material. See the "Sieve and Hydrometer Analysis" graph page for the complete material description

Liquid Limit Test	1	2	3
Tare Number	1882	259	2232
Wt. of Tare & WS (gm)	41.16	41.97	42.73
Wt. of Tare & DS (gm)	34.48	34.31	34.80
Wt. of Tare (gm)	19.25	17.36	17.44
Wt. of Water (gm)	6.7	7.7	7.9
Wt. of DS (gm)	15.2	17.0	17.4
Moisture Content (%)	**43.9**	**45.2**	**45.7**
Number of Blows	**33**	**24**	**18**

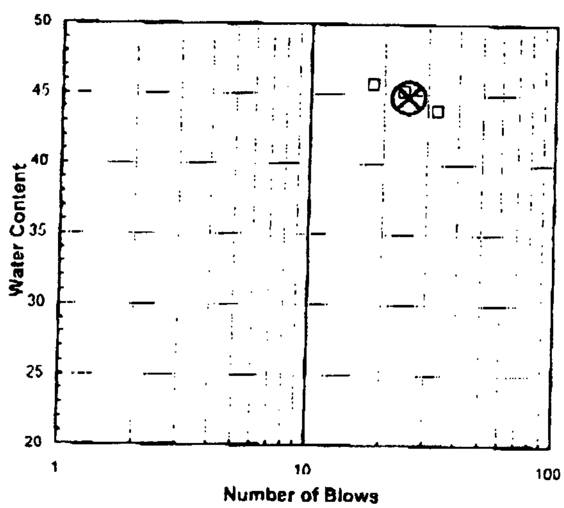
MULTIPOINT

Plastic Limit Test	1	2	Range
Tare Number	2314	2319	
Wt. of Tare & WS (gm)	24.67	24.45	
Wt. of Tare & DS (gm)	23.61	23.41	
Wt. of Tare (gm)	18.25	18.09	
Wt. of Water (gm)	1.1	1.0	
Wt. of DS (gm)	5.4	5.3	
Moisture Content (%)	**19.8**	**19.5**	**0.2**

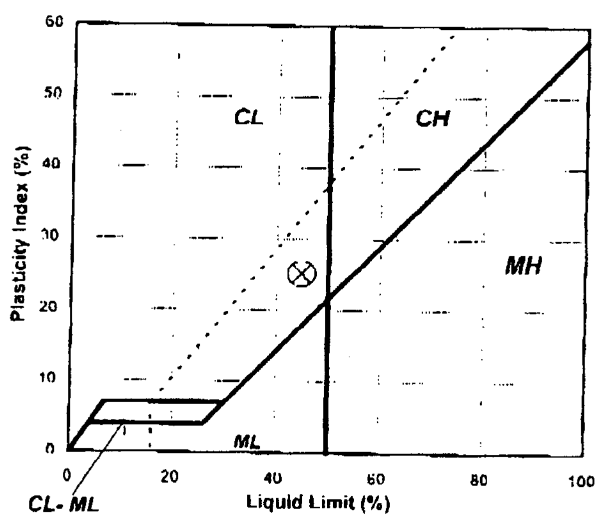
Note: The acceptable range of the two Moisture contents is ± 2.6

Test Results	
Liquid Limit (%)	**45**
Plastic Limit (%)	**20**
Plasticity Index (%)	**25**
USCS Symbol	**CL**

Flow Curve



Plasticity Chart



Tested By BS *Date* 08/30/04 *Checked By* Jcm *Date* 12-2-04

 DCN: CT S4B DATE: 10/08/01 REVISION: 2

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DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

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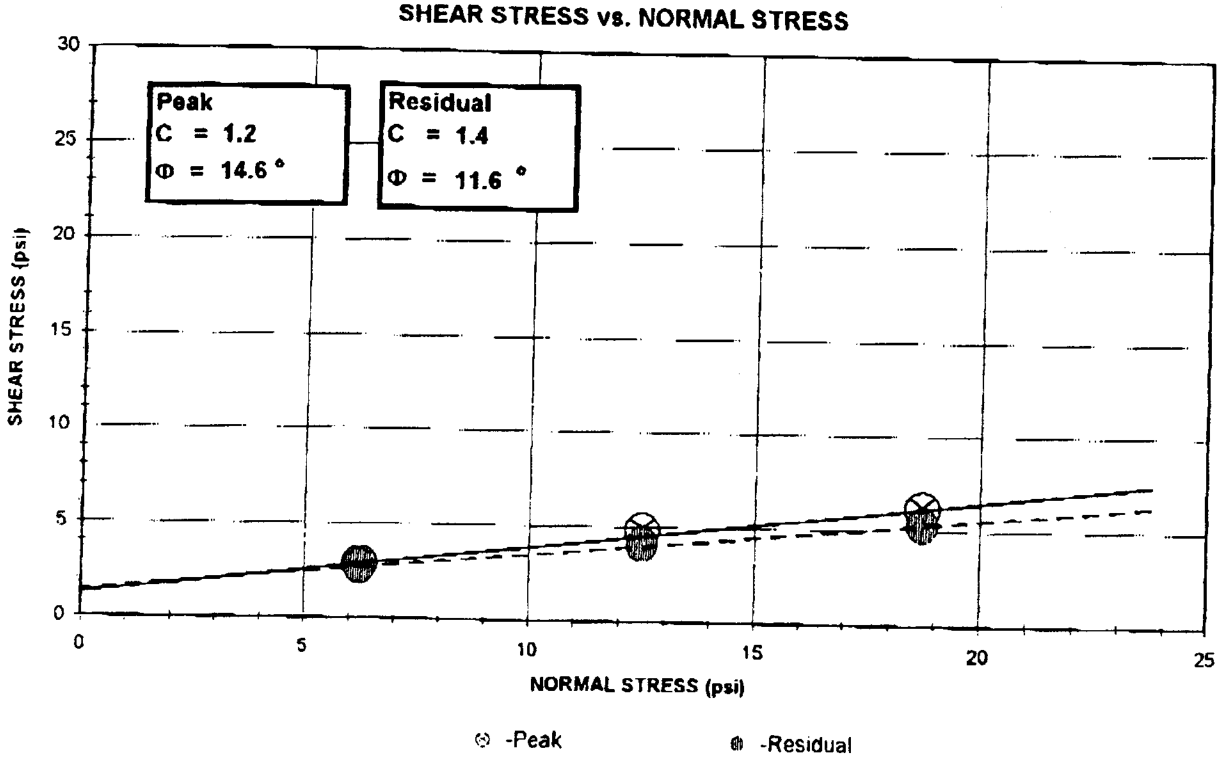
Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

Maximum Shear Stress (psi)		Normal Stress (psi)	PEAK Overall Regression Analysis	Selected Points	PEAK Selected Points Regression
2.85	(1)	6.25	Slope = 0.26		Slope = 0.26
4.81	(2)	12.5	C = 1.3	1	C = 1.2
6.11	(3)	18.75	Φ = 14.6 degrees	3	Φ = 14.6 degrees

Maximum Shear Stress (psi)		Normal Stress (psi)	RESIDUAL Overall Regression Analysis	Selected Points	RESIDUAL Selected Points Regression
2.69	(1)	6.25	Slope = 0.21		Slope = 0.21
4.05	(2)	12.5	C = 1.4	1	C = 1.4
5.26	(3)	18.75	Φ = 11.6 degrees	3	Φ = 11.6 degrees

SHEAR STRESS vs. NORMAL STRESS



Note: Graph not to scale

Tested By TM Date 9/13/04 Approved By DB Date 12/2/04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dshear2004\[M&M2004-237-02-03DSP&R.xls]FINAL PLOT

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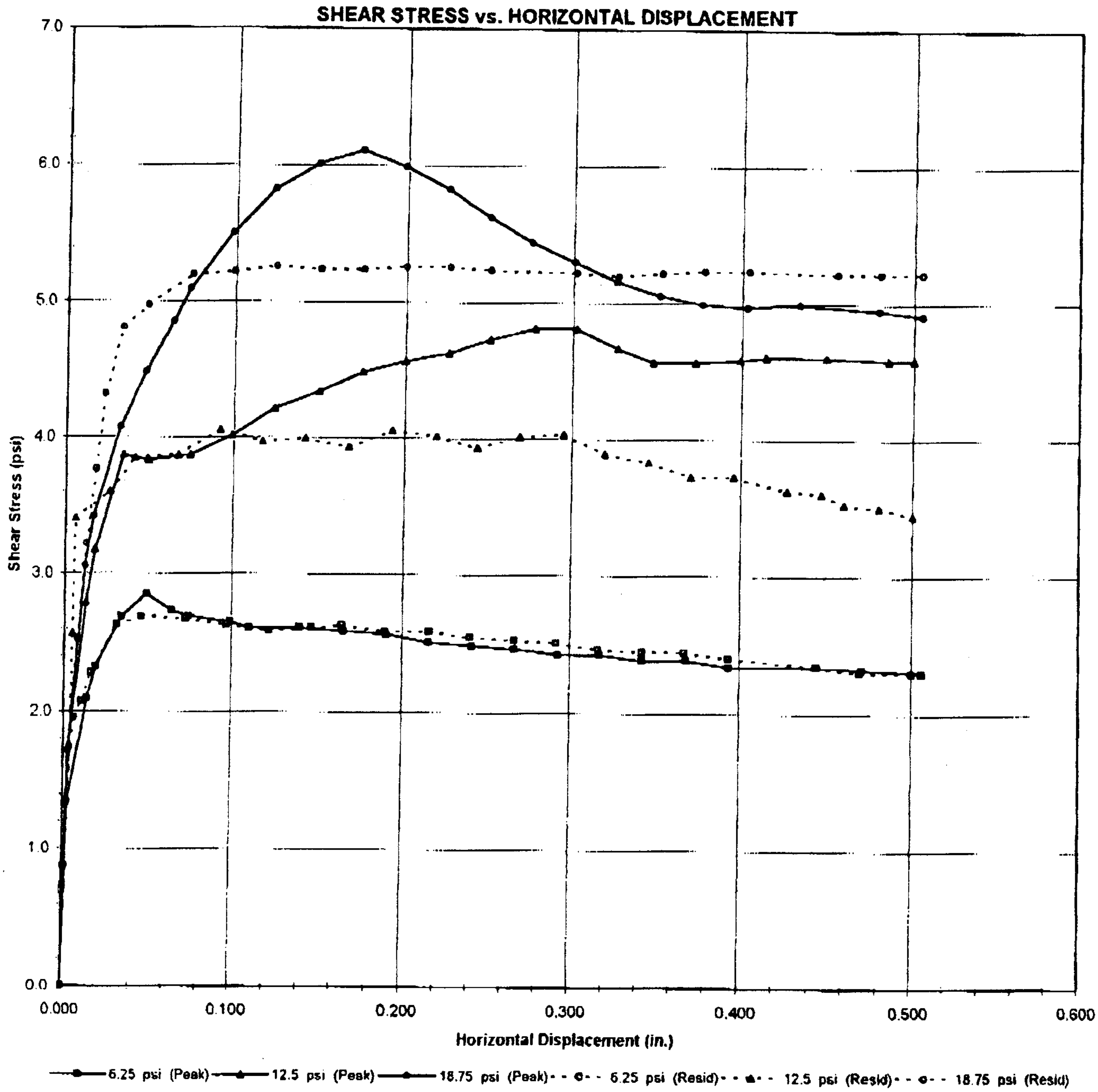
DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

geotechnics INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-01	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED



Tested By TM Date 9/13/04 Approved By DB Date 12/2/04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dshear2004\M&M2004-237-02-03DSP&R.xls]PLOT

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Peak

DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

geotechnics
INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	598.12	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.45	Volume of Solids(cc)	33.5
Weight of Wet Specimen (gm)	140.67	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	393.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	365.3
Wet Density(pcf)	109.2	Void Ratio Before Consolidation	1.40
Dry Density(pcf)	**70.2**	**Void Ratio After Consolidation**	1.31

Moisture Content	Before Test	After Test
Tare ID	1399	1399
Wt. Wet Soil & Tare (gm)	119.48	165.82
Wt. Dry Soil & Tare (gm)	90.45	128.64
Wt. Tare (gm)	38.19	38.19
Wt. of Water (gm)	29.03	37.18
Wt. of Dry Soil (gm)	52.26	90.45
Moisture Content (%)	**55.5**	**41.1**

Testing Parameters	
Normal Stress(psi)	**6.25**
Strain Rate(in/min)	0.00067
Machine Deflection(div)	28

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	**0.00**	0.0	0.0000	0.00
0.001	3.6	**0.73**	0.0	0.0000	0.12
0.003	6.6	**1.34**	0.0	0.0000	0.22
0.014	10.3	**2.10**	14.0	-0.0014	0.34
0.019	11.4	**2.32**	21.0	-0.0021	0.37
0.034	13.2	**2.69**	41.0	-0.0041	0.43
0.049	14.0	**2.85**	55.0	-0.0055	0.46
0.064	13.4	**2.73**	67.0	-0.0067	0.44
0.074	13.2	**2.69**	72.0	-0.0072	0.43
0.099	13.0	**2.65**	84.0	-0.0084	0.42
0.110	12.8	**2.61**	93.0	-0.0093	0.42
0.140	12.8	**2.61**	99.0	-0.0099	0.42
0.166	12.7	**2.59**	107.0	-0.0107	0.41
0.191	12.6	**2.57**	113.0	-0.0113	0.41
0.216	12.3	**2.51**	117.0	-0.0117	0.40
0.242	12.2	**2.49**	122.0	-0.0122	0.40
0.267	12.1	**2.46**	127.0	-0.0127	0.39
0.293	11.9	**2.42**	129.0	-0.0129	0.39
0.318	11.9	**2.42**	134.0	-0.0134	0.39
0.343	11.7	**2.38**	136.0	-0.0136	0.38
0.369	11.7	**2.38**	140.0	-0.0140	0.38
0.394	11.5	**2.34**	144.0	-0.0144	0.37
0.445	11.5	**2.34**	148.0	-0.0148	0.37
0.471	11.4	**2.32**	153.0	-0.0153	0.37
0.506	11.3	**2.30**	156.0	-0.0156	0.37

Tested By TM *Date* 9/2/04 *Input Checked By* BF *Date* 12-2-04

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Peak

DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

geotechnics
INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	602.00	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.42	Volume of Solids(cc)	34.8
Weight of Wet Specimen (gm)	144.58	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	154.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	108.9
Wet Density(pcf)	112.2	Void Ratio Before Consolidation	1.31
Dry Density(pcf)	**73.0**	**Void Ratio After Consolidation**	1.28

Moisture Content	*Before Test*	*After Test*
Tare ID	444	40
Wt. Wet Soil & Tare (gm)	234.24	227.98
Wt. Dry Soil & Tare (gm)	187.27	198.35
Wt. Tare (gm)	99.83	101.55
Wt. of Water (gm)	46.97	29.63
Wt. of Dry Soil (gm)	87.44	96.8
Moisture Content (%)	**53.7**	**30.6**

Testing Parameters	
Normal Stress(psi)	**12.5**
Strain Rate(in/min)	0.00067
Machine Deflection(div)	45

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	**0.00**	0.0	0.0000	0.00
0.001	4.4	**0.90**	0.0	0.0000	0.07
0.003	8.5	**1.73**	1.0	-0.0001	0.14
0.013	13.7	**2.79**	8.0	-0.0008	0.22
0.018	15.6	**3.18**	10.0	-0.0010	0.25
0.034	19.0	**3.87**	12.0	-0.0012	0.31
0.049	18.8	**3.83**	13.0	-0.0013	0.31
0.074	19.0	**3.87**	19.0	-0.0019	0.31
0.099	19.7	**4.01**	21.0	-0.0021	0.32
0.124	20.7	**4.22**	26.0	-0.0026	0.34
0.150	21.3	**4.34**	27.0	-0.0027	0.35
0.175	22.0	**4.48**	28.0	-0.0028	0.36
0.200	22.4	**4.56**	28.0	-0.0028	0.37
0.226	22.7	**4.62**	28.0	-0.0028	0.37
0.251	23.2	**4.73**	29.0	-0.0029	0.38
0.277	23.6	**4.81**	29.0	-0.0029	0.38
0.302	23.6	**4.81**	29.0	-0.0029	0.38
0.327	22.9	**4.67**	29.0	-0.0029	0.37
0.348	22.4	**4.56**	23.0	-0.0023	0.37
0.373	22.4	**4.56**	23.0	-0.0023	0.37
0.399	22.5	**4.58**	21.0	-0.0021	0.37
0.414	22.6	**4.60**	21.0	-0.0021	0.37
0.449	22.6	**4.60**	19.0	-0.0019	0.37
0.485	22.5	**4.58**	19.0	-0.0019	0.37
0.500	22.5	**4.58**	19.0	-0.0019	0.37

Tested By TM *Date* 9/7/04 *Input Checked By* RF *Date* 12-2-04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dshear2004\M&M2004-237-02-03DSPLR.xls]SECOND

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Peak

DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

geotechnics INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	596.91	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.38	Volume of Solids(cc)	35.3
Weight of Wet Specimen (gm)	139.53	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	933.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	877.0
Wet Density(pcf)	108.3	Void Ratio Before Consolidation	1.28
Dry Density(pcf)	**74.0**	**Void Ratio After Consolidation**	1.08

Moisture Content	*Before Test*	*After Test*
Tare ID	1399	444
Wt. Wet Soil & Tare (gm)	117.03	214.11
Wt. Dry Soil & Tare (gm)	92.23	186.20
Wt. Tare (gm)	38.80	99.83
Wt. of Water (gm)	24.8	27.91
Wt. of Dry Soil (gm)	53.43	86.37
Moisture Content (%)	**46.4**	**32.3**

Testing Parameters	
Normal Stress(psi)	**18.75**
Strain Rate(in/min)	0.00067
Machine Deflection(div)	56

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	**0.00**	0.0	0.0000	0.00
0.002	4.3	**0.88**	0.0	0.0000	0.05
0.003	8.4	**1.71**	1.0	-0.0001	0.09
0.012	15.0	**3.06**	19.0	-0.0019	0.16
0.017	16.8	**3.42**	35.0	-0.0035	0.18
0.032	20.0	**4.07**	81.0	-0.0081	0.22
0.047	22.0	**4.48**	123.0	-0.0123	0.24
0.063	23.8	**4.85**	162.0	-0.0162	0.26
0.073	25.0	**5.09**	186.0	-0.0186	0.27
0.098	27.0	**5.50**	240.0	-0.0240	0.29
0.123	28.6	**5.83**	282.0	-0.0282	0.31
0.148	29.5	**6.01**	317.0	-0.0317	0.32
0.174	30.0	**6.11**	345.0	-0.0345	0.33
0.199	29.4	**5.99**	367.0	-0.0367	0.32
0.225	28.6	**5.83**	385.0	-0.0385	0.31
0.250	27.6	**5.62**	400.0	-0.0400	0.30
0.275	26.7	**5.44**	411.0	-0.0411	0.29
0.301	26.0	**5.30**	423.0	-0.0423	0.28
0.326	25.3	**5.15**	434.0	-0.0434	0.27
0.352	24.8	**5.05**	448.0	-0.0448	0.27
0.377	24.5	**4.99**	455.0	-0.0455	0.27
0.403	24.4	**4.97**	461.0	-0.0461	0.27
0.433	24.5	**4.99**	469.0	-0.0469	0.27
0.479	24.3	**4.95**	480.0	-0.0480	0.26
0.505	24.1	**4.91**	484.0	-0.0484	0.26

Tested By TM *Date* 8/28/04 *Input Checked By* BF *Date* 12-2-04

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Residual

DIRECT SHEAR

ASTM D 3080-90 (SOP-S21)

geotechnics INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	598.12	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.45	Volume of Solids(cc)	33.5
Weight of Wet Specimen (gm)	140.67	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	393.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	365.3
Wet Density(pcf)	109.2	Void Ratio Before Consolidation	1.40
Dry Density(pcf)	**70.2**	**Void Ratio After Consolidation**	1.31

Moisture Content	*Before Test*	*After Test*
Tare ID	1399	1399
Wt. Wet Soil & Tare (gm)	119.48	165.82
Wt. Dry Soil & Tare (gm)	90.45	128.64
Wt. Tare (gm)	38.19	38.19
Wt. of Water (gm)	29.03	37.18
Wt. of Dry Soil (gm)	52.26	90.45
Moisture Content (%)	**55.5**	**41.1**

Testing Parameters	
Normal Stress(psi)	**6.25**
Strain Rate(in/min)	0.00067
Machine Deflection(div)	28

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	**0.00**	0.0	0.0000	0.00
0.000	3.4	**0.69**	0.0	0.0000	0.11
0.002	6.5	**1.32**	1.0	-0.0001	0.21
0.006	9.6	**1.96**	2.0	-0.0002	0.31
0.011	10.2	**2.08**	5.0	-0.0005	0.33
0.016	11.2	**2.28**	7.0	-0.0007	0.37
0.031	12.9	**2.63**	11.0	-0.0011	0.42
0.046	13.2	**2.69**	13.0	-0.0013	0.43
0.072	13.1	**2.67**	13.0	-0.0013	0.43
0.097	12.9	**2.63**	16.0	-0.0016	0.42
0.122	12.7	**2.59**	16.0	-0.0016	0.41
0.147	12.8	**2.61**	16.0	-0.0016	0.42
0.165	12.9	**2.63**	16.0	-0.0016	0.42
0.190	12.7	**2.59**	16.0	-0.0016	0.41
0.216	12.7	**2.59**	16.0	-0.0016	0.41
0.241	12.5	**2.55**	16.0	-0.0016	0.41
0.267	12.4	**2.53**	16.0	-0.0016	0.40
0.292	12.3	**2.51**	16.0	-0.0016	0.40
0.317	12.1	**2.46**	16.0	-0.0016	0.39
0.343	12.0	**2.44**	20.0	-0.0020	0.39
0.368	12.0	**2.44**	20.0	-0.0020	0.39
0.394	11.8	**2.40**	20.0	-0.0020	0.38
0.444	11.5	**2.34**	20.0	-0.0020	0.37
0.470	11.3	**2.30**	20.0	-0.0020	0.37
0.500	11.3	**2.30**	20.0	-0.0020	0.37

Tested By TM *Date* 9/5/04 *Checked By* BF *Date* 12-2-04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dsheer2004\[M&M2004-237-02-03DSP&R.xls]Sheet 1

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Residual

DIRECT SHEAR

ASTM D 3080-98 (SOP-S21)

geotechnics
INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	602	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.42	Volume of Solids(cc)	34.8
Weight of Wet Specimen (gm)	144.58	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	154.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	108.9
Wet Density(pcf)	112.2	Void Ratio Before Consolidation	1.31
Dry Density(pcf)	**73.0**	**Void Ratio After Consolidation**	1.28

Moisture Content	*Before Test*	*After Test*	*Testing Parameters*	
Tare ID	444	40		
Wt. Wet Soil & Tare (gm)	234.24	227.98	Normal Stress(psi)	12.5
Wt. Dry Soil & Tare (gm)	187.27	198.35		
Wt. Tare (gm)	99.83	101.55	Strain Rate(in/min)	0.00067
Wt. of Water (gm)	46.97	29.63		
Wt. of Dry Soil (gm)	87.44	96.80	Machine Deflection(div)	45
Moisture Content (%)	53.7	30.6		

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	0.00	0.0	0.0000	0.00
0.004	8.6	1.75	16.0	-0.0016	0.14
0.005	12.6	2.57	21.0	-0.0021	0.21
0.006	16.7	3.40	29.0	-0.0029	0.27
0.026	17.7	3.61	115.0	-0.0115	0.29
0.041	18.9	3.85	155.0	-0.0155	0.31
0.067	19.0	3.87	226.0	-0.0226	0.31
0.092	19.9	4.05	261.0	-0.0261	0.32
0.117	19.5	3.97	276.0	-0.0276	0.32
0.142	19.6	3.99	298.0	-0.0298	0.32
0.168	19.3	3.93	324.0	-0.0324	0.31
0.193	19.9	4.05	347.0	-0.0347	0.32
0.219	19.7	4.01	358.0	-0.0358	0.32
0.244	19.3	3.93	367.0	-0.0367	0.31
0.269	19.7	4.01	412.0	-0.0412	0.32
0.295	19.8	4.03	426.0	-0.0426	0.32
0.320	19.1	3.89	431.0	-0.0431	0.31
0.346	18.8	3.83	436.0	-0.0436	0.31
0.371	18.3	3.73	440.0	-0.0440	0.30
0.396	18.3	3.73	445.0	-0.0445	0.30
0.427	17.8	3.63	449.0	-0.0449	0.29
0.447	17.7	3.61	453.0	-0.0453	0.29
0.460	17.3	3.52	456.0	-0.0456	0.28
0.480	17.2	3.50	456.0	-0.0456	0.28
0.500	16.9	3.44	458.0	-0.0458	0.28

Tested By *TM* *Date* *9/10/04* *Input Checked By* BF *Date* 12-2-04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dshear2004\M&M2004-237-02-03DSP&R.xls]SECOND

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Residual

DIRECT SHEAR

ASTM D 3080-90 (SOP-S21)

geotechnics
INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	B-2
Client Reference	TOWANDA CREEK @ W PHALINGER 04-013	Depth (ft)	22.4-22.8
Project No.	2004-237-02	Sample No.	ST-2
Lab ID	2004-237-02-03	Visual Description	GRAYISH BROWN CLAY

Sample Conditions: UNDISTURBED, INUNDATED AND DOUBLE DRAINED

SHEAR BOX DATA

Wt.of Wet Specimen & Ring(gm)	596.91	Specific Gravity (Assumed)	2.70
Weight of Ring (gm)	457.38	Volume of Solids(cc)	35.3
Weight of Wet Specimen (gm)	139.53	Initial Consolidation Dial Reading	0.0
Initial Specimen Height(in)	1	Final Consolidation Dial Reading	933.0
Specimen Diameter(in)	2.5	Corrected Final Cons. Reading	877.0
Wet Density(pcf)	108.3	Void Ratio Before Consolidation	1.28
Dry Density(pcf)	**74.0**	**Void Ratio After Consolidation**	**1.08**

Moisture Content	Before Test	After Test
Tare ID	1399	444
Wt. Wet Soil & Tare (gm)	117.03	214.11
Wt. Dry Soil & Tare (gm)	92.23	186.20
Wt. Tare (gm)	38.80	99.83
Wt. of Water (gm)	24.80	27.91
Wt. of Dry Soil (gm)	53.43	86.37
Moisture Content (%)	**46.4**	**32.3**

Testing Parameters	
Normal Stress(psi)	**18.75**
Strain Rate(in/min)	0.00067
Machine Deflection(div)	56

Horizontal Displacement (in)	Shear Force (lbs)	Shear Stress (psi)	Vertical Dial Reading 1 div= 0.0001"	Vertical Displacement (+)incr,(-)decr (in)	Shear To Normal Ratio
0.000	0.0	**0.00**	0.0	0.0000	0.00
0.001	4.3	**0.88**	0.0	0.0000	0.05
0.003	8.4	**1.71**	1.0	-0.0001	0.09
0.008	12.4	**2.53**	9.0	-0.0009	0.13
0.013	15.8	**3.22**	17.0	-0.0017	0.17
0.018	18.5	**3.77**	23.0	-0.0023	0.20
0.023	21.2	**4.32**	27.0	-0.0027	0.23
0.033	23.6	**4.81**	33.0	-0.0033	0.26
0.048	24.4	**4.97**	38.0	-0.0038	0.27
0.074	25.5	**5.19**	46.0	-0.0046	0.28
0.099	25.6	**5.22**	47.0	-0.0047	0.28
0.124	25.8	**5.26**	47.0	-0.0047	0.28
0.150	25.7	**5.24**	47.0	-0.0047	0.28
0.175	25.7	**5.24**	47.0	-0.0047	0.28
0.200	25.8	**5.26**	47.0	-0.0047	0.28
0.226	25.8	**5.26**	47.0	-0.0047	0.28
0.251	25.7	**5.24**	47.0	-0.0047	0.28
0.302	25.6	**5.22**	47.0	-0.0047	0.28
0.327	25.5	**5.19**	47.0	-0.0047	0.28
0.353	25.6	**5.22**	47.0	-0.0047	0.28
0.378	25.7	**5.24**	47.0	-0.0047	0.28
0.404	25.7	**5.24**	47.0	-0.0047	0.28
0.455	25.6	**5.22**	46.0	-0.0046	0.28
0.480	25.6	**5.22**	46.0	-0.0046	0.28
0.505	25.6	**5.22**	46.0	-0.0046	0.28

Tested By TM *Date* 8/31/04 *Input Checked By* BF *Date* 12-2-04

 DCN: CT-S21 DATE: 07/20/00 REV: 1 C:\My Documents\Dshear2004\M&M2004-237-02-03DSP&R.xls]THIRD

544 Braddock Avenue • East Pittsburgh, PA 15112 • Phone (412) 823-7600 • Fax (412) 823-8999

GEOTESTING TEST DATA

McMahon & Mann
Consulting Engineers, P.C.

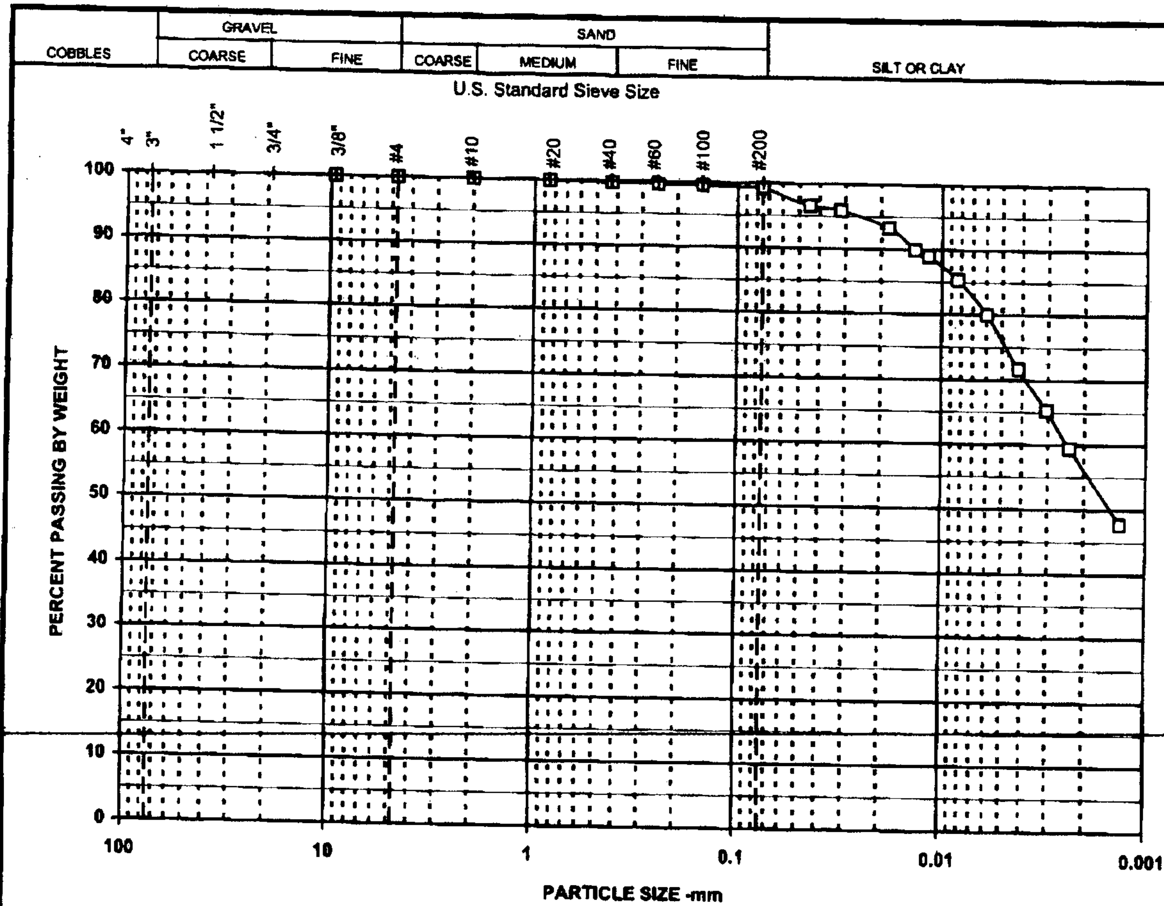
Project No.: 31737826 File: Indx1.xls

LABORATORY TESTING DATA SUMMARY

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS: WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLAS. IND.	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	HYDROMETER % MINUS 2 µm (%)	pH: Distilled Water	pH: 0.01 M CaCl Solution	TOTAL UNIT WEIGHT (pcf)	STRENGTH: Type Test	MAX SHEAR STRENGTH (psf)	REMOLDED SHEAR STRENGTH (psf)	SENSITIVITY
B-2	ST-1	15-17														
B-2	ST-1	15.25	38.1									109.3				
B-2	ST-1	15.9	47.2										LV	188	83	2.3
B-2	ST-1	16.5	49.8										LV	121	59	2.1
B-2	ST-1	17	53.3										LV	385	67	5.7
													LV	315	62	5.1
B-2	ST-3	28-30														
B-2	ST-3	28.2	50.0									112.9				
B-2	ST-3	28.7	44.3										LV	159	48	3.3
B-2	ST-3	29.2	43.0										LV	221	48	4.6
B-2	ST-3	29.8	41.0										LV	194	65	3.0
													LV	210	46	4.6
B-4	ST-2	22.5-24.5														
B-4	ST-2	23.4	52.2									109.0				
B-4	ST-2	23.9	49.8										LV	143	65	2.2
B-4	ST-2	24.4	44.2										LV	320	48	6.7
B-4	ST-2	24.2	56.6										LV	299	46	6.5
													LV	358	54	6.6
		Composite Sample	44.2	46	19	27	CL	99.2	56							

Note:

Prepared by: RV Reviewed by: gl Date: 10/04/2004



Symbol	□	■	○
Boring	Composite		
Sample			
Spec			
Depth			
% +3"			
% Gravel			
% SAND	0.8		
% FINES	99.2		
% -2μ	56		
Cc			
Cu			
LL	46		
PL	19		
PI	27		
USCS	CL		
w (%)			

Particle Size	PERCENT FINER		
(Sieve #)	□	■	○
4"			
3"			
1 1/2"			
3/4"			
3/8"	100.0		
4	100.0		
10	100.0		
20	99.9		
40	99.8		
60	99.7		
100	99.5		
200	99.2		

SYMBOL	DESCRIPTION AND REMARKS
□	Red-Brown CLAY
■	
○	

PARTICLE SIZE DISTRIBUTION
Tonawanda Creek Stabilization

Project No. 31737826	October 2004	Figure

URS Corporation

sieve1a.xls 10/04/2004

Tonawanda Creek Soil Stabilization

SUMMARY OF UNCONFINED COMPRESSION TESTING

SAMPLE NO.	SOIL ADDITIVES Additive Rate (kg/m3)	Cement (%additive)	QuickLime (%additive)	POUR DATE	CURING PERIOD (days)	WATER CONTENT (%)	TOTAL UNIT WGT. (pcf)	DRY UNIT WGT. (pcf)	AXIAL STRAIN @ FAILURE (%)	PEAK COMPRESSIVE STRESS (psi)	REMARKS
Mix 1	75	75	25	10/05/2004	7	43.5	110	76.7	1.2	42.1	UC286b
Mix 1	75	75	25	10/05/2004	14						
Mix 1	75	75	25	10/05/2004	28						
Mix 1	75	75	25	10/05/2004	56						
Mix 2	75	100	0	10/05/2004	7	43.9	109.4	76.1	1.1	52.8	UC286a
Mix 2	75	100	0	10/05/2004	14						
Mix 2	75	100	0	10/05/2004	28						
Mix 2	75	100	0	10/05/2004	56						
Mix 3	50	75	25	10/06/2004	7	44.8	109.6	75.7	1.1	36.4	UC287b
Mix 3	50	75	25	10/06/2004	14						
Mix 3	50	75	25	10/06/2004	28						
~~Mix 3~~	~~50~~	~~75~~	~~25~~	~~10/06/2004~~	56						
Mix 4	50	100	0	10/06/2004	7	43.4	110.4	77	0.8	47.5	UC287a
Mix 4	50	100	0	10/06/2004	14						
Mix 4	50	100	0	10/06/2004	28						
Mix 4	50	100	0	10/06/2004	56						

Prepared by: RV Reviewed by: ________ Date: 10/14/2004

APPENDIX E

SLOPE STABILITY ANALYSES
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

McMahon & Mann
Consulting Engineers, P.C.

APPENDIX E

SLOPE STABILITY ANALYSES
TONAWANDA CREEK ROAD SLOPE STABILIZATION
CLARENCE, NEW YORK

MMCE evaluated the stability of Tonawanda Creek Road for both shallow and deep failure conditions to simulate the observation that initially a shallow failure occurred that was followed by a deep much larger failure. We also evaluated the stability of remedial measures including placing riprap at the toe of the slope to stabilize it, improving the soft clay soil beneath the road embankment using the dry mix method and moving the road away from the slope to improve the stability.

We completed slope stability analyses using the computer program PCSTABL5M developed by Purdue University in conjunction with the Indiana Department of Transportation. PCSTABL5M solves slope stability problems utilizing a two dimensional limit method of slices. Methods of analysis available with this program include the Bishop Method for analysis of circular shaped failure surfaces, the Janbu Method for analysis of general shaped failure surfaces and Spencer's Method for the analysis of circular or general shaped failure surfaces.

We selected the soil properties used in the analyses based on the results of the field and laboratory tests and our experience and judgement.

Original Failure Conditions

Our evaluation of the original conditions of the slope includes both shallow and deep failure conditions. The analysis depicted on the "Original Conditions – Shallow Failure," includes a shear strength for the soft clay of 350 pounds per square foot (17.0 kPA), approximately the average of the peak strength values measured in the Vane Shear Tests (VST's). The analysis also considers the groundwater level in the silty sand deposit to be at the ground surface. The analyses for this condition indicate a factor of safety less than unity. This supports the hypothesis that a build up of water pressure in the silty sand deposit led to the shallow failure initially observed at the site.

The analysis depicted on "Original Conditions – Deep Failure," include a shear strength for the soft clay of 92 pounds per square foot (4.4 kPA), approximately the average of the remolded strength values measured in the VST's. The analyses for this condition indicates a factor of safety less than unity, demonstrating that after the soft clay soils were disturbed, they did not have sufficient strength to support the road embankment. The failure limits shown on the analysis coincide with the location of the scarp and the toe of the failure observed in the field. As indicated on the plot, the analysis indicates that the failure extends to the bottom of the soft clay layer.

Riprap Remediation Analysis

We considered placing riprap or a reinforced earth wall to support the toe and slope of the road embankment as done at some other locations along Tonawanda Creek Road. The analysis depicted on "Rip-Rap Remediation Analysis" indicates a factor of safety less than unity for failure surfaces extending beneath the riprap or reinforced earth zone.

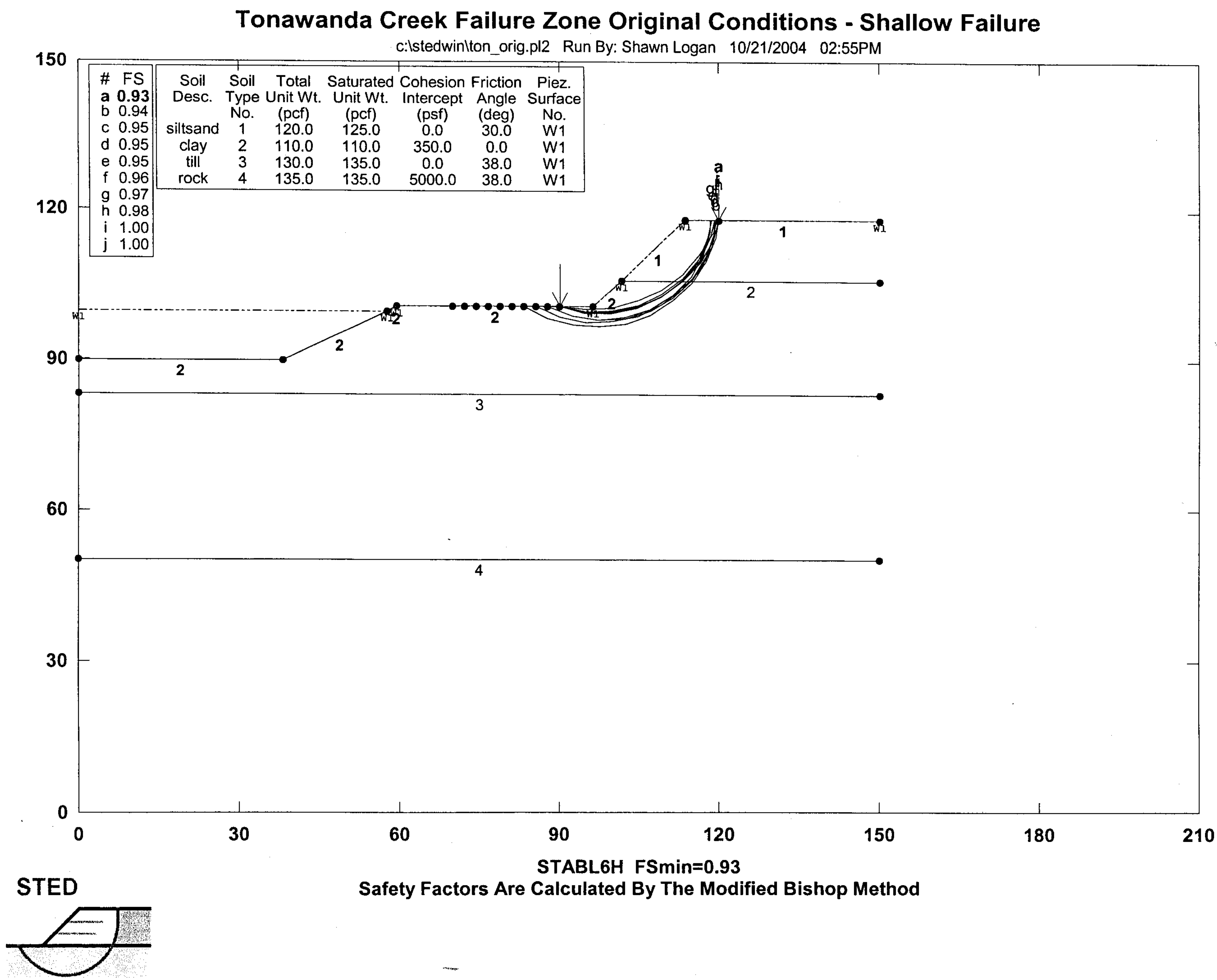
Dry Soil Mix Remediation Analysis

We completed slope stability analyses to evaluate the degree of soil improvement required for the dry soil mixing alternative. The analysis depicted on "Dry Soil Mix Remediation Analysis," indicates a factor of safety of more than 1.5 for an average shear strength in the improved zone of 1200 psf (57.5 kPA).

Flatten Slope, Relocate Road Analysis

We completed slope stability analyses for a revised, flattened slope considering the soft clay. For this analysis "Slope 7H:1V," with the soft clay at the average peak strength of 350 psf (17.0 kPA) the slope would have to be flattened to 7 horizontal to 1 vertical for a factor of safety of 1.5. We also considered an analysis with the soft clay at the average remolded strength of 92 psf (4.4 kPA). For this analysis, a factor of safety of 1.5 is not achieved even for a slope of 10 horizontal to 1 vertical. These analyses demonstrate that relocating the road to the south will not result in the same factor of safety as improving the soil and keeping the road in its present location.

SLOPE STABILITY PLOTS

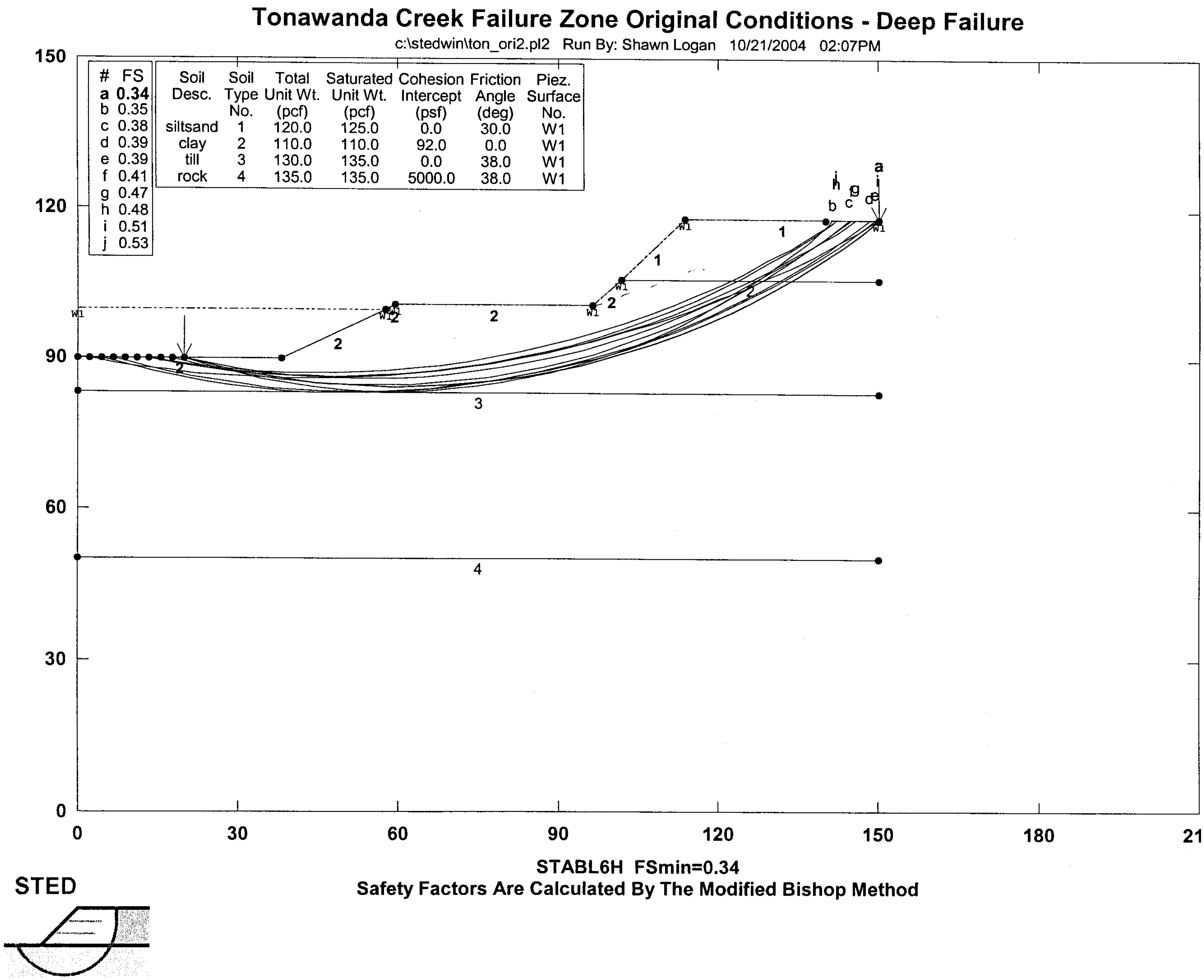


Tonawanda Creek Failure Zone Original Conditions - Shallow Failure
c:\stedwin\ton_orig.pl2 Run By: Shawn Logan 10/21/2004 02:55PM

#	FS
a	**0.93**
b	0.94
c	0.95
d	0.95
e	0.95
f	0.96
g	0.97
h	0.98
i	1.00
j	1.00

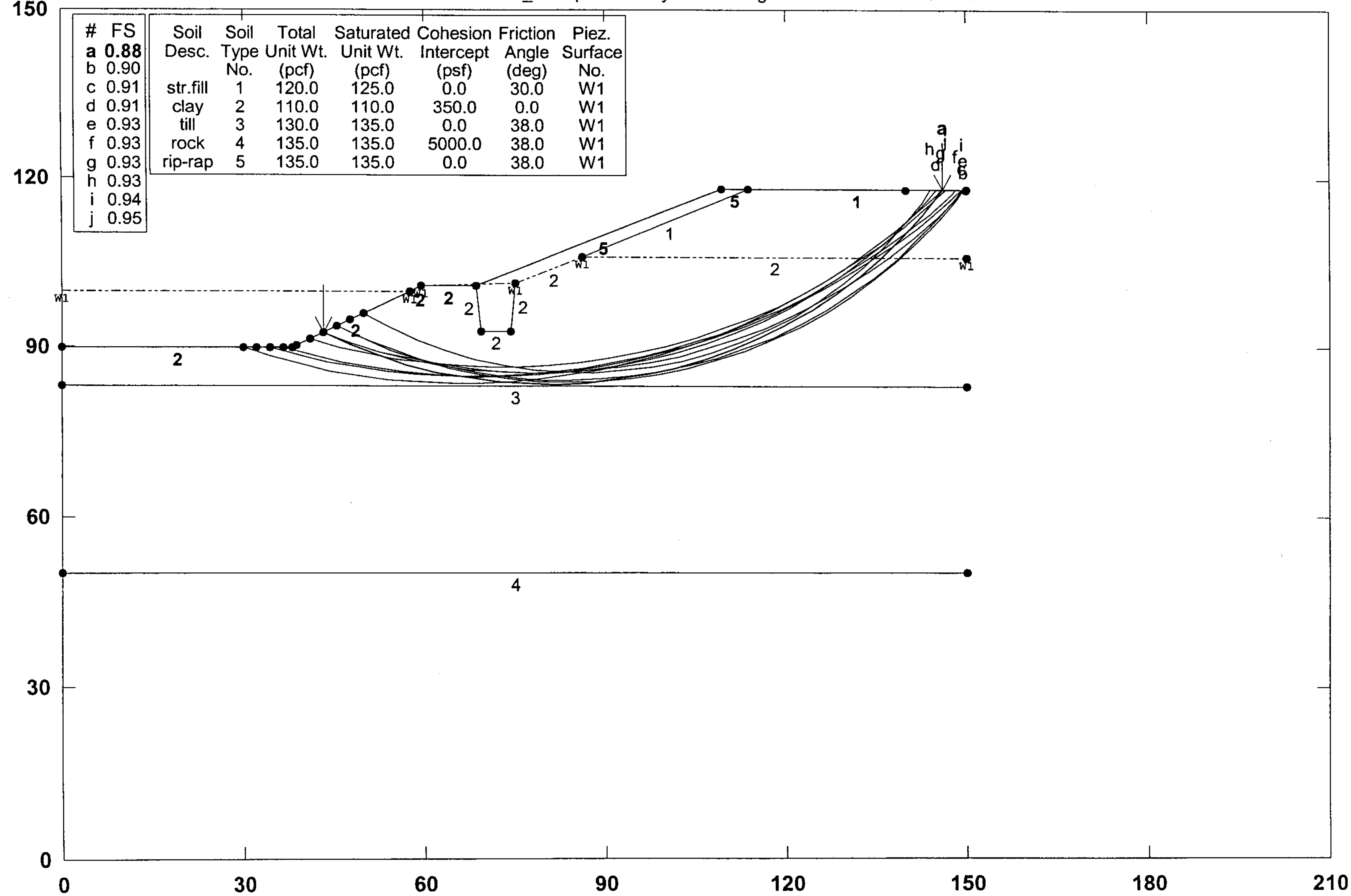
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
siltsand	1	120.0	125.0	0.0	30.0	W1
clay	2	110.0	110.0	350.0	0.0	W1
till	3	130.0	135.0	0.0	38.0	W1
rock	4	135.0	135.0	5000.0	38.0	W1

150
120
90
60
30
0
0
30
60
90
120
150
180
210
STABL6H FSmin=0.93
Safety Factors Are Calculated By The Modified Bishop Method
STED


Tonawanda Creek Failure Zone Original Conditions - Deep Failure
c:\stedwin\ton_ori2.pl2 Run By: Shawn Logan 10/21/2004 02:07PM
FS
a 0.34
b 0.35
c 0.38
d 0.39
e 0.39
f 0.41
g 0.47
h 0.48
i 0.51
j 0.53
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
siltsand 1 120.0 125.0 0.0 30.0 W1
clay 2 110.0 110.0 92.0 0.0 W1
till 3 130.0 135.0 0.0 38.0 W1
rock 4 135.0 135.0 5000.0 38.0 W1
150
120
90
60
30
0
0
30
60
90
120
150
180
210
STABL6H FSmin=0.34
Safety Factors Are Calculated By The Modified Bishop Method
STED

Tonawanda Creek Failure Zone Rip-Rap Remediation Analysis

c:\stedwin\ton_des1.pl2 Run By: Shawn Logan 10/21/2004 02:57PM



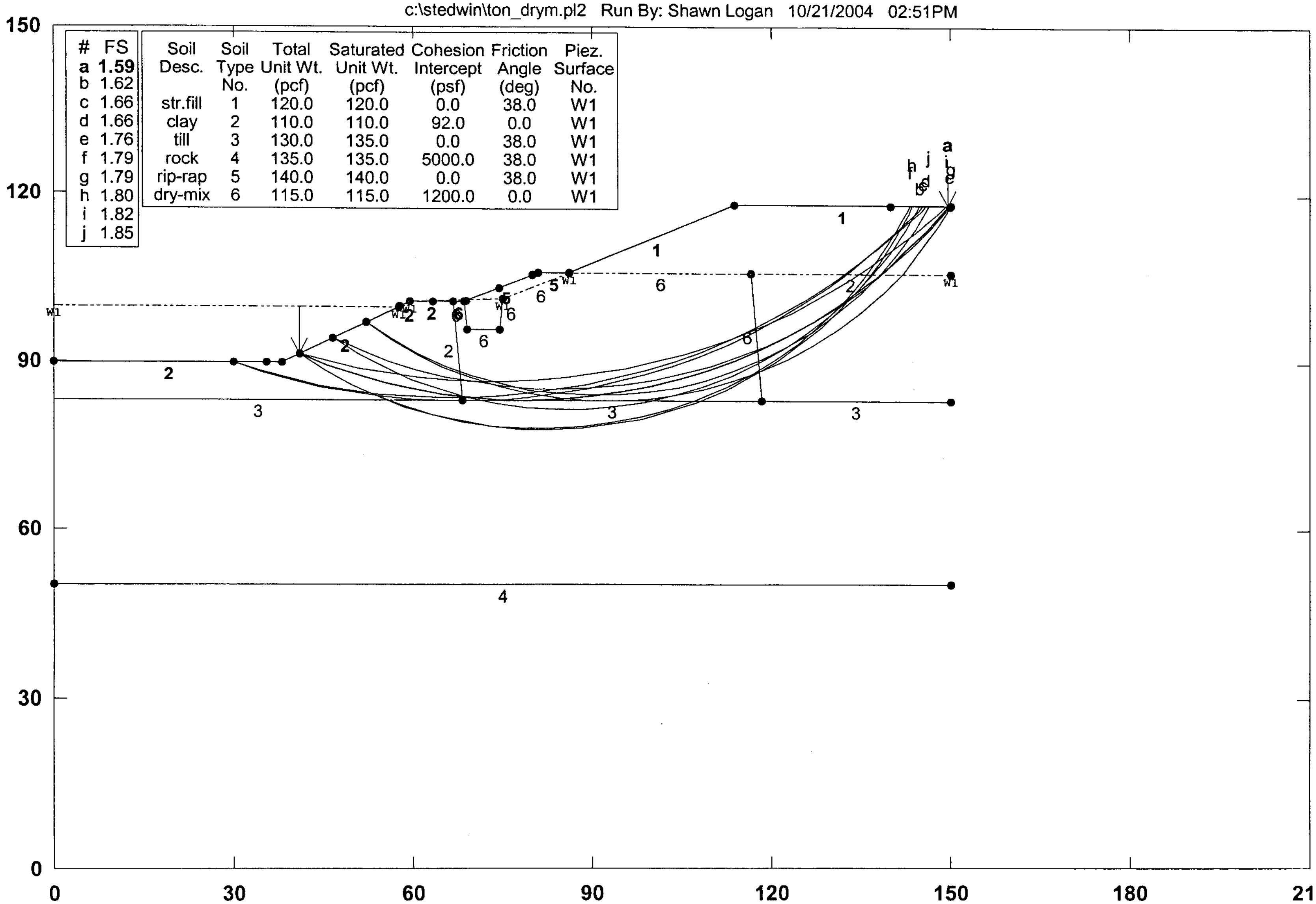
STABL6H FSmin=0.88

Safety Factors Are Calculated By The Modified Bishop Method

STED

Tonawanda Creek Failure Zone Dry Soil Mix Remediation Analysis

c:\stedwin\ton_drym.pl2   Run By: Shawn Logan   10/21/2004   02:51PM



#	FS
a	**1.59**
b	1.62
c	1.66
d	1.66
e	1.76
f	1.79
g	1.79
h	1.80
i	1.82
j	1.85

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
str.fill	1	120.0	120.0	0.0	38.0	W1
clay	2	110.0	110.0	92.0	0.0	W1
till	3	130.0	135.0	0.0	38.0	W1
rock	4	135.0	135.0	5000.0	38.0	W1
rip-rap	5	140.0	140.0	0.0	38.0	W1
dry-mix	6	115.0	115.0	1200.0	0.0	W1

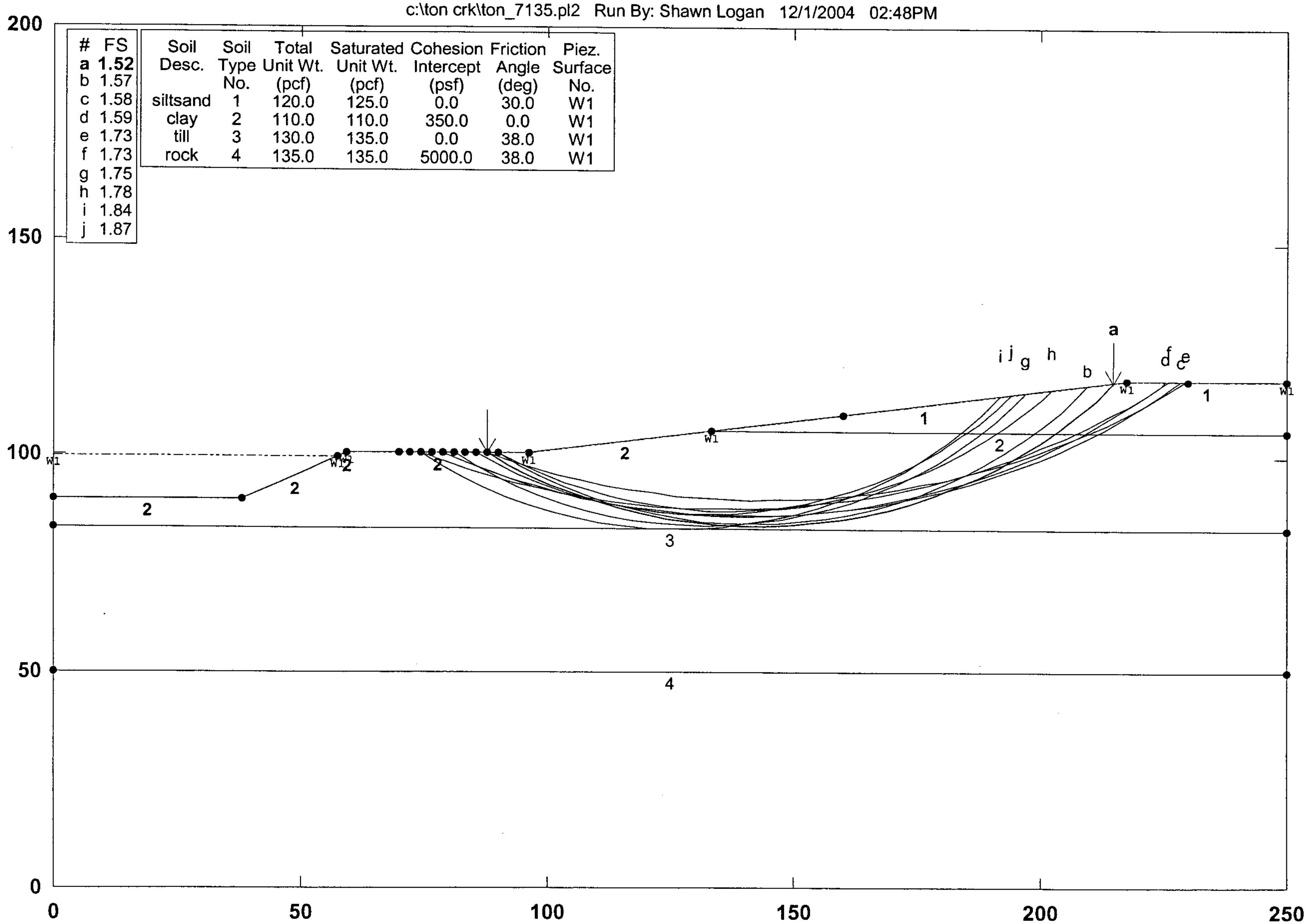
STABL6H FSmin=1.59

Safety Factors Are Calculated By The Modified Bishop Method

STED

Tonawanda Creek Failure Zone Slope 7H : 1V

c:\ton crk\ton_7135.pl2 Run By: Shawn Logan 12/1/2004 02:48PM



STABL6H FSmin=1.52

Safety Factors Are Calculated By The Modified Bishop Method

STED